PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

JORDAN DAM
WILSALL, MONTANA
PARK COUNTY
MT 334

prepared for

HONORABLE TED SCHWINDEN
GOVERNOR, STATE OF MONTANA

THE ARTHUN BROTHERS
(OWNER-OPERATOR)

prepared by

HKM ASSOCIATES
BILLINGS, MONTANA

APRIL 1981

SEATTLE DISTRICT
UNITED STATES
ARMY CORPS OF ENGINEERS

PHASE I INSPECTION REPORT

NATIONAL DAM SAFETY PROGRAM

JORDAN DAM

WILSALL, MONTANA

PARK COUNTY

MT 334

PREPARED FOR:

HONORABLE TED SCHWINDEN
GOVERNOR, STATE OF MONTANA

THE ARTHUN BROTHERS
(OWNER-OPERATOR)

PREPARED BY:

HKM ASSOCIATES, BILLINGS, MONTANA

APRIL 1981

SEATTLE DISTRICT

UNITED STATES
ARMY CORPS OF ENGINEERS

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

EXECUTIVE SUMMARY

Personnel of HKM Associates, under a contract with the Montana Department of Natural Resources and Conservation (DNRC), and with representation from the DNRC and Soil Conservation Service (SCS), inspected Jordan Dam on June 26, 1980. The inspection and evaluation were performed under the authority of Public Law 92-367. Jordan Dam was built in 1961 by the Shields River Ranch Company, with technical assistance provided by the SCS. The Arthun brothers of Wilsall, Montana are the current owners of the project.

FINDINGS AND EVALUATION

Jordan Dam has an upstream contributory drainage area of 3.3 square miles but the primary water supply is an irrigation canal from the Shields River. The dam is located on a tributary of Antelope Creek, approximately 7.7 miles north of Wilsall, Park County, Montana.

Jordan Reservoir is a single-purpose irrigation storage project. Storage capacity to the principal spillway crest is estimated to be 640 acre-feet (AF). Storage between the principal spillway crest and the emergency spillway crest amounts to 100 AF. Total estimated storage capacity to the first overtopping dam crest elevation is 1260 AF. The dam has a hydraulic height of 35 feet. On the basis of criteria in the U.S. Army Corps of Engineers' Recommended Guidelines for Safety Inspection of Dams (Ref. 1), the project is intermediate in size. There is a sufficient number of inhabitable structures in the downstream flood plain that a dam failure could endanger many lives. In addition there are county and private roads, private utilities, irrigation facilities, and farmland that would be affected in the event of a dam breach. The downstream hazard potential is therefore high (Category 1). However, no dam breach analysis or routing of a dam breach flood was made for the downstream area. The conclusions on probable damage are based on a brief field inspection and engineering judgment.

The guidelines recommend that the discharge and/or storage capacity of an intermediate-size, high downstream hazard potential dam be capable of safely handling the Probable Maximum Flood (PMF). The PMF is the flood expected from the most severe combination of meteorologic and hydrologic conditions that are reasonably possible in the region. Assuming that the irrigation canal which intercepts the drainage basin has no effect in reducing peak and volume amounts because of its small size, the estimated PMF for Jordan Dam has a peak discharge of 20,720 cubic feet per second (cfs) and a total 72-hour volume of 2270 AF.

For the flood routing, the initial reservoir pool was assumed to be at the principal spillway crest elevation (61.0 feet, see note on page viii for datum description), and the low-level outlet was assumed closed. Routing of the estimated PMF for Jordan Dam showed that the project has the capacity of controlling a flood having ordinates approximately equal to 81 percent of the PMF hydrograph ordinates.

The low-level outlet and principal spillway did not receive a complete field evaluation due to the pool level at the time of survey, the inability to control the principal spillway flow, and the fact that the pipes are too small to allow inspection with available equipment. The gate was partially operated during the field inspection. No operating problems were evident, even though the 2-1/2 inch galvanized steel casing for the gate stem is broken approximately 3 feet downslope from the concrete pedestal. The section of corrugated steel pipe which is exposed at the outlet end is beginning to show some deterioration of the asphalt coating, but the metal is still in good condition. Erosion and bank sloughing was evident in the stilling basin.

Seepage areas downstream of the embankment were identified during the field investigation. Seepage was evident throughout the valley floor immediately downstream, with the majority of seepage exiting downstream of the left abutment. Free water was evident in some locations. Specific details relative to the phreatic surface throughout the embankment are unknown due to a limited informational base and the fact that field measurements could not be made.

The results of the surficial examination of Jordan Dam generally indicate that the embankment is in good and stable condition. Areas of concern include loss/lack of riprap in the stilling basin and on the upstream face, and some topsoil sloughing on the downstream face. Embankment stability questions still exist due to the lack of sufficient information to evaluate the dam. The dam drain system is apparently not functioning properly.

Because the project is incapable of controlling the full PMF without overtopping and causing the dam to fail, Jordan Dam does not meet inspection guidelines (Ref. 1).

RECOMMENDATIONS

Immediately develop, implement, and test a downstream warning plan, and implement a more active maintenance plan. Reshape the upstream face of the dam above the berm and along the natural shoreline of the right abutment to the proper slope, and provide a weather-resistant rock riprap on the total upstream face where necessary. Suggested improvements of the outlet stilling basin are reshaping and the placement of rock riprap throughout the

basin. Recommended work on the low-level outlet include: identify the cause of the gate stem casing break and perform the necessary corrective measures; repair the gate stem casing and refill with oil; improve the foundation of the concrete pedestal; inspect the low-level outlet pipe throughout its total length when the pool is at a low level, and make the necessary repairs before raising the pool; operate the gate in the absence of principal spillway flows, and check the gate for full travel capability when the reservoir is at a low level; and perform the necessary repairs to the stilling basin as noted above. Perform a complete inspection of the 24-inch riser pipe when the reservoir pool is at an accommodating level, with special attention to the concrete wearing surface at the bottom of the riser. Provide emergency closure capability under any conditions of the principal spillway conduit at the riser. Install piezometers in the embankment, abutments, and foundation, and establish a regular monitoring program to evaluate piezometric conditions. Minor debris maintenance is required on the upstream face, particularly near the conical trash rack on the riser. Repair the left abutment contact on the downstream face where a small erosion gully has developed.

Conduct more detailed hydrologic and hydraulic routing studies to better determine the downstream hazard and required spillway capacity and modify the project as studies indicate. Evaluate embankment structural stability and modify the dam as studies indicate. Conduct periodic inspections of the project at 5-year intervals by engineers experienced in dam design and construction. Develop and implement a periodic maintenance plan for the dam and appurtenant structures.

Prior to performing engineering studies and remedial construction, coordinate the work with the Montana DNRC, Dam Safety Section, to insure compliance with all pertinent laws and regulations.

Mike Keene

Michael D. Keene
Professional Engineer

MONTANA
MICHAEL D.
KEENE
4197 E
REGISTERED
PROFESSIONAL ENGINEER

PERTINENT DATA SUMMARY

1. General

Federal ID Number	MT 334
Owner and Operator	Arthun brothers
Purpose	Irrigation
Location	Section 17, T4N, R9E, MPM; 7 miles north of Wilsall, MT
County, State	Park County, Montana
Watershed	Tributary to Antelope Creek
Hazard Potential	Category 1 (High)
Size Classification	Intermediate

2. Reservoir

Surface Area at Emergency Spillway Crest Elevation 62.3 feet (see note page viii)	87 acres (estimated)
Dead Storage to Low-Level Inlet Elevation 38.7 feet	3 acre-feet
Storage to Principal Spillway Crest Elevation 61.0 feet	640 acre-feet
Storage to Emergency Spillway Crest Elevation 62.3 feet	740 acre-feet
Total Storage to First Overtopping Dam Crest Elevation 67.5 feet	1260 acre-feet
Total Drainage Area	3.3 square miles
Reservoir Water Surface Elevation on the Day of the Inspection	61.3 feet

PERTINENT DATA SUMMARY
(Continued)

3. Principal Spillway

Crest Elevation	61.0 feet
Type	Unregulated, 24-inch diameter, 14 gauge corrugated steel pipe (CSP) riser with conical trash rack cover. Riser connects to 18-inch diameter CSP discharge pipe.
Spillway Capacity to First Overtopping Dam Crest Elevation 67.5 feet	27 cubic feet per second

4. Emergency Spillway

Crest Elevation	62.3 feet
Type	Unregulated, trapezoidal earth spillway
Crest Width	240 feet
Spillway Capacity to First Overtopping Dam Crest Elevation 67.5 feet (Including Adjacent Overland Flow)	11,140 cubic feet per second

5. Low-Level Outlet

Gate	18-inch diameter slidegate valve
Control	Manual operator
Conduit	194 feet of 18-inch diameter corrugated steel pipe. By specification, it is 14 gauge, close riveted and asphalt coated.

PERTINENT DATA SUMMARY
(Continued)

Capacity To Principal Spillway Crest Elevation 61.0 feet	18 cubic feet per second

6. Dam

Type	Earth fill
Structural Height	38 feet
Hydraulic Height	35 feet
Design Crest Control Elevation	67.0 feet
Existing First Overtopping Dam Crest Elevation	67.5 feet
Crest Length	650 feet
Design Dam Crest Width	14.0 feet
Existing Dam Crest Width	Approximately 14 feet
Upstream Dam Slope	1V on 3H (above berm elevation 61.0 feet) 1V on 4H (below berm according to plans)
Downstream Dam Slope	1V on 1.6H

Note: The June 26, 1980 field survey used the principal spillway crest elevation of 61.0 feet (obtained from the construction plans) as datum. Basis for construction plans' datum unknown.

CHAPTER 1
BACKGROUND

1.1 INTRODUCTION

1.1.1 Authority

This report summarizes the Phase I inspection and evaluation of Jordan Dam. The project is owned and operated by the Arthun brothers, Wilsall, Montana.

The National Dam Inspection Act, Public Law 92-367 dated August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to conduct safety inspections of non-federal dams throughout the United States. Pursuant to that authority, the Chief of Engineers issued "Recommended Guidelines for Safety Inspection of Dams" in Appendix D, Volume 1 of the U.S. Army Corps of Engineers' Report to the United States Congress on "National Program of Inspection of Dams" in May 1975.

The recommended guidelines were prepared with the help of engineers and scientists highly experienced in dam safety from many federal and state agencies, professional engineering organizations and private engineering consulting firms. Consequently, the evaluation criteria presented in the guidelines represent the comprehensive concensus of the engineering community.

Where necessary, the guidelines recommend a two-phased study procedure for investigating and evaluating existing dam conditions so deficiencies and hazardous conditions can be readily identified and corrected. The Phase I study is:

(1) a limited investigation to assess the general safety condition of the dam
(2) based upon an evaluation of the available data and a visual inspection
(3) performed to determine if any needed emergency measures and/or if additional studies, investigations and analyses are necessary or warranted
(4) not intended to include extensive explorations, analysis, or to provide detailed alternative correction recommendations.

The Phase II investigation includes all additional studies necessary to evaluate the safety of the dam. Included in Phase II, as required, should be additional visual inspections, measurements, foundation exploration and testing, material testing, hydraulic and hydrologic analyses and structural stability analyses.

The authority for the Corps of Engineers to participate in the inspection of non-federally owned dams is limited to Phase I investigations with the exception of situations of extreme emergency. In these cases the Corps may proceed with Phase II studies but only to the extent needed to answer serious questions relating to dam safety that cannot be answered otherwise.

The two phases of investigations outlined above are intended only to evaluate project safety and do not encompass in scope the engineering required to perform design or corrective modification work. Recommendations contained in this report may be for either Phase II safety analyses or detailed design study for corrective action.

The responsibility for implementation of these Phase I recommendations rests with the State of Montana, Department of Natural Resources and Conservation. The owner/operator is urged to contact the Montana DNRC prior to taking any action on report recommendations. It should be noted that nothing contained in the National Dam Inspection Act, and no action or failure to act under this Act, shall be construed (1) to create liability in the United States or its officers or employees for the recovery of damage caused by such action or failure to act or (2) to relieve an owner or operator of a dam of the legal duties, obligations, or liabilities incident to the ownership or operation of the dam.

The investigation process allows for report review by: the Montana DNRC; the Soil Conservation Service (SCS); and the Arthun brothers (owner/operator). Review comments are considered before final publication of the Phase I Inspection Report. Their written comments are enclosed in Appendix E.

1.1.2 Purpose and Inspection

The findings and recommendations in this report were based on visual inspection of the project, minimal field survey measurements, and review of available design and operation data. The purpose of the inspection is to make a general assessment as to the structural integrity and operational adequacy of the dam embankment and its appurtenant structures. Inspection procedures and criteria were those established by the Recommended Guidelines for Safety Inspection of Dams (Ref. 1).

The visual inspection of Jordan Dam was made on June 26, 1980. HKM Associates personnel who attended the field inspection and contributed to this report were:

Dan Dyer, Geotechnical Engineer
Gary Elwell, Hydraulics/Hydrology
Mike Keene, Hydraulics/Hydrology, Team Leader

Other HKM personnel contributing to the report but not attending the field inspection were:

Dale R. Cunnington, Structural Engineer
William Hansen, Hydraulics/Hydrology
Dan Nebel, Geology

Other personnel present during the June 26, 1980 inspection included:

Glen McDonald, Supervisor, Montana DNRC, Dam Safety Section
Larry Tegg, Dam Safety Engineer, Montana DNRC, Dam Safety Section
Dave Jones, Soil Conservation Service
Glenn Malmquist, Soil Conservation Service

1.2 DESCRIPTION

1.2.1 General

Jordan Dam is an earth fill dam located in the W 1/2 of Section 17, T4N, R9E, M.P.M., Park County, Montana (Appendix A and Ref. 2, 3, 4). The Arthun brothers own and operate the project.

The project forms an irrigation facility within the Missouri River Basin and is fed primarily by a canal from the Shields River. Jordan water enters Antelope Creek 500 feet downstream of the dam. Antelope Creek travels approximately 1.8 miles before joining the Shields River (Appendix A and Ref. 2). The nearest downstream community is Wilsall, Montana, which is located on the west bank of the Shields River approximately 7 miles south of the dam. In terms of a stream channel distance, Wilsall is located about 8 miles downstream of the dam. There are several homes located in the flood plain of the Shields River.

Jordan Dam has a hydraulic height of 35 feet and impounds 1260 AF at the first overtopping dam crest elevation (elevation 67.5 feet, see note on page viii). Based on a visual reconnaissance and engineering judgment, at least three residences, as well as roads, bridges, irrigation facilities, and agricultural land will be affected by a sudden breach of the dam. On the basis of this information and in accordance with the Recommended Guidelines (Ref. 1), the project is classified intermediate in size and the downstream hazard potential is high (Category 1).

Jordan Dam was constructed as an irrigation storage facility. The dam has an emergency spillway, a principal spillway and a low-level outlet. The emergency spillway is trapezoidal in shape, with a base width of 240 feet and side slopes of 1V on 4H, and is located 600 feet south of the right dam abutment contact. The principal spillway consists of a 24-inch diameter corrugated

steel pipe (CSP) riser and an 18-inch diameter CSP discharge pipe. The low-level outlet is 18 inches in diameter and is incorporated with the principal spillway discharge pipe. Storage to the normal pool level, or the principal spillway crest (elevation 61.0 feet), is 640 AF (Ref. 5). Flood storage of 100 AF is available between the principal spillway crest and the emergency spillway crest (elevation 62.3 feet). An additional 520 AF are available for flood surcharge storage between the emergency spillway crest and the first overtopping dam crest elevation.

Jordan Dam has a total upstream contributory drainage area of 3.3 square miles, but the primary water supply is a canal from the Shields River. The Jordan watershed is a prairie drainage (Photo 1 of Appendix B). Elevations in the basin range from 6000 feet NGVD to about 5380 feet NGVD at the reservoir (Ref. 4).

1.2.2 Regional Geology

Jordan Dam is situated near the north end of the Shields River valley in central Montana. The topography at the damsite is one of a shallow alluvial valley. Structurally, the dam lies near the north end of the axis of the Crazy Mountain syncline and is bordered by the Crazy Mountain uplift to the east, the Bridger Mountain uplift to the west, and the Smith River-Shields River divide to the north. The strata in the area shows a uniform southwesterly dip of 30 to 40 degrees, with the Livingston Formation forming the surface of the area. Numerous small faults are present in the immediate area, however, there is no evidence of recent activity (Ref. 5, 6).

1.2.3 Seismicity

Jordan Dam is in a moderately active seismic zone with the majority of the region's seismic events occurring in the Southwestern Montana-Yellowstone Park area. Since 1925, Montana has experienced five shocks that reached intensity VIII or greater (Modified Mercalli Scale). The closest epicenter occured in the Three Forks, Montana area which is approximately 42 miles west of the damsite. Numerous other shocks of intensity IV or greater have been reported within a 100-mile radius of the site (as of January 1974). The site is located in zone 3 of the Seismic Zone Map of Contiguous States, and it can be assumed that a major earthquake may occur within the life of the structure (Ref. 1, 7). Although the Zone Map is based on a known distribution of damaging earthquakes, it does not necessarily reflect accurate or adequate seismic design parameters for this site.

1.2.4 Site Geology

H.R. Smith of the SCS completed a geologic reconnaissance of the dam and reservoir site in August 1960 (Ref. 5). In addition, numerous exploration holes were completed along the dam axis, pool area, and emergency spillway area prior to construction (Sheet 2 of Exhibit C1). This section of the report is developed from these sources plus personal knowledge of the area.

The reservoir basin is formed in alternating beds of interbedded sandstone and sandy shale of the Livingston Formation. The strike and dip could not be determined for the bedrock outcrop in the immediate area, however, the regional bedding has a strike to the northeast and the dip is shallow to the southwest (downstream direction). The alternating sandstone and shale layers vary in thickness from 1 foot to 20 feet. The shale readily undergoes decomposition when exposed to the air. The valley section consists of alluvial and colluvial deposits of gravel, sand, and silt over weathered shale and sandstone. The maximum depth of the alluvial deposit is over 30 feet.

The outlet pipe and stilling basin are located in alluvium. During the field inspection it was noted that erosion is occurring along the stilling basin perimeter. The stilling basin erosion has progressed downward to bedrock, a depth of about 4 feet.

1.2.5 Design and Construction History

Jordan Dam was built by the Shields River Ranch Company in 1961 as an irrigation storage facility. Technical design assistance was provided by the SCS. The SCS also provided construction inspection and quality control. The construction contractor is unknown. Project funding was shared by the Shields River Ranch Company and the government (Ref. 5). The Arthun brothers, Wilsall, Montana are the current owners of the project.

Design of Jordan Dam is reported to have been accomplished in the design section of the SCS State office located in Bozeman, Montana. Construction supervision was provided by SCS work unit personnel out of the Livingston, Montana office (Ref. 5).

Test hole logs were developed prior to construction, with the results being placed on the construction plans and in the design file. Laboratory tests, including moisture-density relationships and grain-size analyses, were performed by the SCS on selected samples taken from the borrow areas and test holes. The results of these tests are available in the design file. Soil strength tests were apparently not performed (Ref. 5).

Little information is available relative to construction inspection diaries. It was noted, however, that a minor problem was experienced during project construction concerning contractor performance. A letter from an SCS field representative on October 13, 1961 indicated that the contractors were not complying with the specifications, primarily with regard to the required compaction. The SCS threatened not to certify the project as per the specifications, which would have eliminated financial participation by the government. SCS project involvement continued subsequent to the October 13 letter, which would seem to imply that satisfactory performance was obtained by the SCS field personnel (Ref. 5). The SCS indicates that twelve in-place density tests were made on the cove backfill and embankment (Appendix E).

No records of post-construction maintenance and repairs are available for review. The owner indicates that major problems have not been experienced to date.

CHAPTER 2
INSPECTION AND RECORDS EVALUATION

2.1 HYDRAULICS AND STRUCTURES

2.1.1 Spillway

The spillway system for Jordan Dam consists of a principal and emergency spillway. A detailed description of each follows.

2.1.1.1 Principal Spillway

The principal spillway is located approximately 400 feet from the left abutment contact, is aligned perpendicular to the centerline of the dam, and lies approximately in the valley bottom. It consists of a 24-inch diameter, 14 gauge corrugated steel pipe (CSP) riser with 18-inch diameter, 14 gauge tee sections welded at the base. In addition, there is approximately 110 lineal feet of 18-inch diameter, 14 gauge CSP and a stilling pool.

The riser is unregulated and has a conical trash rack and an antivortex plate. The 24-inch diameter riser stands approximately 24 feet above the 18-inch CSP discharge pipe and rests on a 4'x4'x12" reinforced concrete pad. (Sheet 3 of Exhibit C1).

The principal spillway is an extension of the low-level outlet system. All pipe is close riveted and asphalt dipped, and is connected with watertight bands. The pipe rests on natural ground and is covered with "trench backfill". Antiseep collars are located on a variable spacing throughout the impervious zone material. According to the plans the pipe was placed on a uniform grade of 2.52 percent (Sheet 3 of Exhibit C1). A detailed discussion of the outlet pipe is presented in Section 2.1.2.

The principal spillway riser and the 18-inch diameter discharge pipe were not totally inspected during the Phase I investigation because the reservoir pool level was too high and the appropriate equipment was not available to inspect the small diameter pipe. A surficial examination was made of the riser inlet and the pipe outlet, and a detailed inspection of the downstream channel was performed.

Energy dissipation is provided by a small stilling pool. The pipe outlet was designed as a cantilevered system. The pool has a width and depth of about 12 feet and 3.5 feet respectively. Erosion has created vertical walls along the upstream side of the stilling basin (Photo 2 of Appendix B). There is no evidence of riprap in the pool. Downstream of the pool the channel transitions to a base width of about 4-1/2 feet. The banks are

approximately 3 feet high and essentially vertical. The flow enters Antelope Creek about 500 feet downstream of the dam.

There was a small amount of water entering the principal spillway riser (crest elevation 61.0 feet, see note on page viii) and passing through the discharge pipe at the time of the inspection. The riser appears to be operating satisfactorily, although the antivortex plate is not resting upright on the riser pipe (Photo 3 of Appendix B).

Principal spillway rating information was not available; therefore, new hydraulic rating information was developed. The spillway system was analyzed for weir, orifice, and pipe control. For friction calculations, an "n" value of 0.025 was chosen and it was assumed the pipe flows full except for low flows. The full pipe flow assumption is reasonable despite low tailwater elevations because pipe control exists except for low flows. The principal spillway discharge is 27 cfs to the first overtopping elevation (67.5 feet). It appears the spillway is capable of passing these flows without distress, except for possible damage to the return channel. Discharge rating information is provided in Exhibits D2 and D3 of Appendix D.

2.1.1.2 Emergency Spillway

The emergency spillway is located approximately 600 feet south of the right dam abutment contact. The spillway is trapezoidal in shape with a base width of 240 feet and cut slopes of 1V on 4H. There is a 50-foot control section located approximately 45 degrees to the dam axis. The control section is leveled to elevation 62.3 feet according to the June 26, 1980 survey. The spillway is constructed in soils of an erodible nature, but erosion in the spillway channel should not endanger the dam embankment since the two are separated by about 600 feet. The spillway outlets in the Antelope Creek Valley approximately 700 feet downstream from the toe of the dam (Sheet 1 of Exhibit C1). The spillway is covered with natural grasses and weeds 6 to 12 inches high, which should not provide much resistance to hydraulic flows. Towards the center and north side of the spillway, the weeds are coarser and higher, with some sagebrush at the spillway brink (Photo 4 of Appendix B).

The original design criteria for the emergency spillway size and dam crest height was not available for review in the files. In addition, the original downstream hazard assessment and structure rating by the SCS is unknown. In general, it can be said that the spillway width and dam crest height were established according to SCS standards and guidelines which were applicable at the time of design. New hydraulic rating information was

developed because no emergency spillway rating information was available. Calculations were based on the weir head-discharge equation and a Mannings "n" value of 0.04. Under maximum surcharging conditions to the first dam overtopping elevation (67.5 feet), the emergency spillway capacity is estimated to be 8200 cfs.

The construction plans indicate an optional dike to the left of the emergency spillway (Sheet 1 of Exhibit C1). Based on field investigation, this optional dike was never constructed. The dam crest profile (Exhibit C2 of Appendix C) shows a low area just to the left of the emergency spillway which is capable of passing flood flows above elevation 64.2 feet. The discharge capability of this area was estimated by using the weir head-discharge formula and a representative rectangular flow area 240 feet wide. This area also consists of erodible materials, but there is approximately 360 feet of distance to the dam embankment and, therefore, erosion should not be a problem. The discharge rating information has been combined with that of the emergency spillway and is provided in Exhibits D2 and D3 of Appendix D. The combined discharge at the first dam overtopping elevation (67.5 feet) is 11,140 cfs.

2.1.2 Low-Level Outlet

A low-level outlet gate and pipe are provided to allow complete drawdown of the reservoir pool and provide irrigation water releases. The outlet pipe is incorporated with the principal spillway pipe. The inlet consists of a trash rack, an 18-inch slidegate, and a reinforced concrete structure having an inlet elevation of 38.7 feet (Sheet 3 and 4 of Exhibit C1). The concrete support pedestal and handwheel operator are located on a local berm and positioned such that it does not interfere with the principal spillway riser (Photo 3 of Appendix B). The inlet structure is connected to the bottom of the spillway riser by approximately 85 lineal feet of 18-inch CSP. Similar to the CSP downstream of the riser, the upstream pipe was designed to have a 14-gauge thickness and was to be close riveted and asphalt dipped. The pipe lengths are connected utilizing watertight bands. Pipe slope from the inlet section to the riser is 2.52 percent. The low-level outlet system and principal spillway utilize a common 18-inch CSP discharge pipe for the remaining distance under the embankment, and a common outlet facility. The pipe upstream of the riser rests on natural ground and is covered with "trench backfill", just as the pipe downstream of the riser.

The 2.5-inch galvanized steel pipe which encloses the gate stem is broken at a threaded connection and is allowing water to enter

(Photos 3 and 5 of Appendix B). Wave erosion of the gravel and hard shale, used as riprap on the upstream face, uncovered the gate stem pipe and undermined the pedestal. The fracture of the pipe was likely due to ice pressure and pedestal settlement. This is designed to be a closed system containing lubrication for the gate stem. The gate was partially opened during the inspection, and the low-level outlet system was operated for a short period of time. The water inside the steel pipe has not significantly harmed gate operation yet, but could become a problem in the future. The outlet pipe and gate were not inspected due to the high reservoir pool level and lack of appropriate equipment to inspect the small diameter conduit. It was not possible to tell if a complete seal of the gate can be obtained due to flow over the principal spillway crest. See Section 2.1.1 for a discussion of the stilling pool.

Low-level outlet rating information was not available. New hydraulic rating information was developed assuming the gate is fully open, a Manning's "n" value of 0.025, and full pipe flow exists except for very low discharges. The low-level outlet discharge at the principal spillway crest elevation (61.0 feet) is 18 cfs.

2.1.3 Freeboard

Flood routing (Section 2.2.3) indicates the dam overtops during the guidelines' (Ref. 1) recommended spillway design flood (which is the full PMF), and therefore, no freeboard exists for such conditions. The vertical distance from the principal spillway crest (elevation 61.0 feet) to the first overtopping dam crest elevation (67.5 feet) is 6.5 feet. The project operator indicated that a probable historic highwater level for Jordan Dam occurred either in 1975 or 1979. The maximum pool level on either of these dates was unknown; however, the project operator indicated that if the spillway passed flows, they were relatively small. The vertical distance between the reservoir pool and the first overtopping elevation at the time of the June 1980 field inspection was 6.2 feet.

Jordan Reservoir is basically oriented in a north-south direction, with the dam on the south side. The prevailing wind for this region is generally identified as being westerly (Ref. 8, 9). The reservoir location and orientation can be observed in Appendix A. The effective fetch length is calculated to be 0.5 mile. Hence, a minimum freeboard allowance should be about 3 feet (Ref. 10). This 3-foot allowance does not apply to an event as severe as the PMF where the design objective would be to provide only enough freeboard to maintain structural integrity of the dam. Although the dam will overtop during the PMF and lesser floods, the vertical distance between normal pool and dam crest is sufficient to prevent overtopping by wind-generated waves.

2.2 HYDROLOGY

2.2.1 Physiography and Climatology

The 3.3-square mile catchment area above Jordan Reservoir is basically rectangular in shape. In particular, the drainage area is approximately 3 miles long and only 1 mile wide. The topography includes flat plains and rolling prairies. The basin is composed of Big Timber-Castner Soils and terraced soils developed in outwash materials from the adjacent mountains, chiefly the Crazy Mountains (Ref. 11). The immediate vicinity of the reservoir can primarily be characterized as prairie drainage. There is spring activity in the drainage basin (Ref. 2). An irrigation canal, which feeds the reservoir, traverses the upstream shore of the reservoir. It is assumed that at the time of the flood event the canal will be flowing full and not provide any detention of flood runoff.

The regional climate is classified as distinctly continental, and characterized by abundant sunshine, low relative humidity, light rainfall, and wide daily and seasonal variations in temperature. However, the regional climate does not have the extreme variable pattern common to the more mountainous western sections in Montana. In general, the valleys are relatively dry during the colder months and wet during the late spring and early summer. The wettest part of the year in the mountains is generally from midwinter to early spring. It is not uncommon for the region to experience winter warming spells with associated thawing temperatures. Precipitation at weather station Wilsall 8ENE (9 miles southeast of the dam) is about 15.5 inches. The average annual temperature at Wilsall 8ENE is approximately 42 degrees Fahrenheit. Winters are typically cold, with January being the coldest month. The monthly average temperature for January at Wilsall 8ENE is about 25 degrees Fahrenheit. During the summer, July is typically the warmest month with an average temperature of about 64 degrees Fahrenheit (Ref. 8, 9). It is assumed that saturated, not frozen ground conditions would be present during a typical late spring, early summer rainfall/flood runoff event. No inflow, outflow or pool level records on Jordan Reservoir are available. The original SCS hydrologic design criteria and hazard classification for Jordan Reservoir was unavailable for review in the project files.

2.2.2 Estimated Probable Maximum Flood (PMF)

The probable maximum precipitation (PMP) and the estimated probable maximum flood were developed for the Jordan drainage basin. The ratio of the reservoir area to the non-reservoir area is 4 percent; therefore, the two areas were separated for the

purpose of this analysis. Although the primary water supply is a canal from the Shields River, it is assumed that the canal contribution will be small relative to the contribution from the drainage basin during a late spring, early summer rainfall/flood runoff event.

The PMF is the flood that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the study region.

Jordan Dam is located west of the 105th meridian and east of the continental divide, hence PMP values were calculated using procedures described in the National Weather Service's Interim Method (Ref. 12). The Interim Method provides PMP values for durations of 6 to 72 hours, with PMP values for 1 to 5 hours obtained by multiplying the 6-hour PMP value by percentages supplied by the Corps of Engineers, Seattle District. The PMP 6-hour, 12-hour, 24-hour, 48-hour, and 72-hour values are 10.9 inches, 13.5 inches, 16.2 inches, 18.8 inches and 21.1 inches, respectively. The 6-hour PMP value of 10.9 inches was multiplied by 67 percent, 78 percent, 85 percent, 91 percent, and 95 percent to obtain PMP values of 7.3 inches, 8.5 inches, 9.3 inches, 9.9 inches and 10.4 inches for durations of 1-hour, 2-hours, 3-hours, 4-hours, and 5-hours, respectively.

The 6-hour increments for the total 72-hour storm were arranged in a critical distribution using criteria presented in the National Weather Services's Hydrometeorological Report No. 43 (Ref. 13). In particular, the 6-hour rainfall increments were arranged according to pattern "e". Further subdivision of the calculated rainfall increments was required to provide compatibility with the duration of the unit hydrograph. A 10-minute unit hydrograph was chosen for Jordan Dam using criteria presented in the SCS Hydrology Handbook (Ref. 14). The unit hydrograph was developed for the Jordan basin using the SCS method and the U.S. Army Corps of Engineers' computer program HEC-1 (Ref. 14, 15). The PMP storm was plotted in the form of a depth-duration curve for convenience in selecting incremental rainfall values. The peak 10-minute PMP values, within the time base of the unit hydrograph, were ordered according to the reverse pattern of the unit hydrograph ordinates.

Rainfall losses were assumed equal to the minimum soil retention rate of 0.15 inches/hour for type B soils (SCS classification system) due to an assumed saturated ground condition.

The runoff condition, or PMF, resulting from a PMP storm was estimated using the PMP values and the unit hydrograph approach. The resultant PMF has a peak flow of 20,720 cfs and a 72-hour volume of 2270 AF.

2.2.3 Flood Routing

The PMF resulting from the PMP rainfall/runoff event was routed through the reservoir using the computer program HEC-1 (Ref. 15). For the purpose of determining a starting elevation for the PMF, an antecedent storm was considered. A 100-year 24-hour rainfall event was chosen as an antecedent condition due to its severity and possible occurrence during late spring and early summer. It is assumed that average soil moisture conditions would be present prior to the antecedent storm. The antecedent runoff hydrograph was developed using SCS techniques. It was assumed that the PMF would begin 24 hours after the initiation of the antecedent storm. The starting reservoir pool level for the PMF event, determined by routing the antecedent runoff through the reservoir, was determined to be 61.5 feet (see note on page viii).

Reservoir elevation-capacity data was obtained from the plans. The elevation-capacity curve was extended to elevation 70 feet by planimetering the 60, 65, and 70 foot contours of the topographic map provided in the plans. Results are provided in Exhibit D1 of Appendix D. Jordan discharge rating data is shown in Exhibits D2 and D3 of Appendix D. Flow through the low-level outlet pipe was not included in the flood routing because the operating gate would probably be closed during a typical late spring, early summer rainfall/flood runoff event.

For the purposes of flood routing, and according to Phase I investigation criteria, the minimum dam crest elevation is the elevation at which overtopping of the dam will occur. This criteria assumes overtopping and failure of embankment-type dams to be coincidental. Based upon the embankment profile survey dated June 26, 1980 (Exhibit C2 of Appendix C), the existing low-point dam crest elevation is 67.5 feet. There is a low area (elevation 64.2 feet) adjacent to the emergency spillway, but it is about 400 feet away from the dam embankment and would not contribute to a dam embankment failure. Therefore, the low point of 67.5 feet was chosen on the dam embankment.

Flood routing showed that the dam would first overtop during the PMF when approximately 68 percent of the total PMF volume enters the reservoir. Routings were made of lesser hypothetical floods than the PMF to determine the magnitude of floods the dam can contain. The hypothetical hydrographs are obtained by applying percentages to the PMF ordinates. A flood with a hydrograph having ordinates corresponding to 81 percent PMF ordinates is just controlled by the project. Larger floods would overtop the dam.

2.3 GEOTECHNICAL EVALUATION

The geotechnical evaluation of Jordan Dam included a field investigation and a search and review of project data. The field inspection consisted of photo documentation, a dam crest profile survey, slope stability observations of the dam embankment, seepage observations, and measurements of the slope angles. Inspection photos are included in Appendix B, some of the construction plans are included in Exhibit C1 of Appendix C, and the crest profile survey is shown in Exhibit C2. Results of the slope angle measurements are provided in Exhibit C3.

2.3.1 Dam Embankment

Description

Jordan Dam is an earth fill structure which was completed in 1961. The dam has an estimated maximum structural height above the deepest point on the foundation surface of 38 feet and a crest length of 650 feet. A 240-foot wide spillway is located approximately 600 feet south of the right abutment contact. Based on field evidence and the surveyed profile (Exhibit C2), the optional dike shown on the construction plans (Sheet 1 of Exhibit C1) was apparently never constructed. The crest width was designed to be at least 14 feet (Photo 6 of Appendix B), and approximate field measurements verify this width.

The construction plans (Sheet 3 of Exhibit C1) indicate the upstream face of the dam has a slope of 1V on 3H down to an 8-foot wide berm located at elevation 61.0 feet (see note on page viii), then flattens to 1V on 4H. Field measurements verified the slope above the berm, but due to the pool level, the slope below the berm could not be verified. The upstream face below the berm was designed with a 36-inch cover of loose gravel and hard shale. Damages resulting from a lack of sufficient riprap protection are discussed in Section 2.3.3. There is a moderately dense cover of grasses, weeds, and sagebrush on the upstream slope above the June 26, 1980 water surface (Photo 7 of Appendix B). Debris on the upstream face is limited to very small material, which is expected because of the nature of the drainage area.

The plans indicate the downstream face of the dam has a slope of 1V on 2H (Sheet 3 of Exhibit C1). A field measurement indicated an angle of 32 degrees or about 1V on 1.6H on the downstream face. There is a moderately dense cover of natural grasses, weeds, and sagebrush on the downstream slope (Photo 8 of Appendix B).

Details provided on Sheet 3, Exhibit C1 indicate the embankment cross section consists of class B-2 fill material. The construction specifications describe this material as clayey-silts (ML-CL Unified Classification) with a maximum dry density of 105 pounds per cubic foot and an optimum moisture of 21 percent. A maximum particle size of 6 inches was allowed (Ref. 5). A detailed description of the as-built material is not available. Except for the filter and riprap materials, which are described in Section 2.3.4, the embankment is apparently class B-2 throughout.

The construction plans also indicate that a shallow core trench with a minimum base width of 8 feet is located on centerline at the base of the embankment (Sheet 2 of Appendix C1). The plans indicate the cutoff trench is about 7.5 feet deep. The final depth of the cutoff was to be determined by the field engineer. The actual depth of the trench is unknown. The plans suggest that the cutoff trench does not extend to bedrock throughout.

There is an old earthen dike located approximately 100 yards downstream of the dam embankment (Photo 9 of Appendix B). This dike suggests that an old reservoir may have been located immediately downstream of the existing embankment or that the Jordan Dam may be located in the pool area of an older reservoir. The impact of this old reservoir on the design and construction of Jordan Dam is unknown. It is also possible that this dike was constructed as a retaining dike on the right side of Antelope Creek.

<u>Settlement</u>

The construction plans indicate that the dam crest control was established at elevation 67.0 feet. An embankment profile was surveyed during the field investigation (Exhibit C2 of Appendix C). The survey shows that the maximum differential elevation along the dam crest is about 1.0 feet. This differential elevation is primarily the result of camber. The settlements are small and are considered insignificant.

2.3.2 Foundation Conditions and Seepage Control

Foundation Conditions

Thirteen (13) test hole logs are available for review on the construction plans (Sheet 2 of Appendix C1). Additional logs were available for review in the design file (Ref. 5).

Laboratory tests were performed by the SCS on selected samples taken from the borrow areas and test holes. These tests included

moisture-density relationships and a grain-size analysis. The results of these tests are available in the design file (Ref. 5). Soil strength tests were apparently not performed.

Based on available information, it appears that the dam embankment is founded primarily on alluvial deposits of sand, gravel, and clay. The alluvial deposits are underlain by a fractured, pervious, laminated sandstone and shale. Bedrock is very shallow at the left abutment but relatively deep in the valley floor (greater than 10 feet) and at the right abutment (30 feet).

Seepage Control

The construction plans indicate underdam seepage was to be controlled by a drain system installed at the base of the embankment (Sheet 2 of Exhibit C1) and by a cutoff trench. The plans indicate this drain is a 2-foot thick gravel section consisting of coarse filter material encased in a 3-inch thick pitrun sand and gravel blanket. A 6-inch diameter, perforated, 16 gauge corrugated metal pipe was placed in the coarse gravel filter medium below elevation 40.0 feet. The gravel filter system extends across the valley to elevation 46.0 feet on the abutments.

Based on the drill logs, it is likely that the cutoff trench under the embankment does not extend to bedrock throughout (Sheet 2 of Exhibit C1).

A mechanical analysis of gravel, proposed as the coarse filter material, is available in the design file (Ref. 5). This material does not meet the construction specification requirements but was considered satisfactory by the design engineer for this purpose. It was suggested that the fine filter medium be aggregate meeting the requirements of ASTM C-33. It is not known what materials were actually used in construction of either the coarse or fine filter sections.

At the time of the June 1980 field investigation, the Jordan Dam pool level was at elevation 61.3 feet. Seepage was evident throughout the valley floor immediately downstream from the dam. However, the majority of seepage was coming out of the left abutment. This seepage area on the left abutment (Photos 8 and 10 of Appendix B) was identified by saturated soil and lush water-type grasses. Free water was evident at some locations. Some seepage was exiting at the toe of the embankment just above the upper end of the drain system on the left abutment.

The seepage control system which includes the cutoff trench and drain did not appear to be functioning properly as considerable seepage was passing under the dam and very little seepage was exiting from the filter drain collection pipes (Photo 2 of Appendix B). However, some of the downstream seepage may be from water passing under the dam through the laminated bedrock dipping to the southwest.

2.3.3 Stability

Embankment

The slope angles measured during the field investigation are shown in Exhibit C3 of Appendix C. These angles were measured with an Abney level and should be considered approximate.

There is some loss/lack of riprap on the upstream slope (Photo 7 of Appendix B) and the stilling basin, erosion of the natural soil in the right abutment due to wave action (Photo 11 of Appendix B), and sloughing of the topsoil on the downstream slope. However, there is no outward sign of embankment instability. The phreatic surface in the embankment is unknown. Available design information suggests that soil strength tests were not performed and stability calculations were not available (Ref. 5).

Small scarps are evident on the downstream face at several locations where topsoil has sloughed. The depth of the slope sloughing material is shallow, probably less than about 1.5 feet. This condition is not considered of immediate danger. This topsoil sloughing on the downstream face may develop into a stability and/or erosion problem in time.

Erosion

Embankment and channel bank erosion were observed at several locations. Without corrective measures, these points of erosion may develop into stability problems.

At isolated locations on the upstream face of the embankment, wave action has eroded the embankment to low vertical cuts leaving the slope exposed and oversteepened (Photos 6 and 7 of Appendix B).

Bank erosion was observed in the natural soils of the right abutment (Photo 11 of Appendix B) and the natural alluvial soils in the stilling basin (Photo 2 of Appendix B). This erosion is the result of wave action undercutting the shoreline and causing sloughing up to about 3 feet in vertical height.

The reservoir shorelines are considered to be in stable condition as no major slides or scarpments were observed. The pool shoreline is occasionally vertical, or near vertical, to heights generally less than 2 feet, and localized sloughing occurs due to wave action and saturated conditions.

The construction specifications required density control during construction of the dam embankment. The SCS indicates that twelve in-place density tests were performed on the cove backfill and embankment (Appendix E). A brief history of construction progress is also available in the design file (Ref. 5).

2.3.4 Rock Riprap

A riprap blanket has been placed on a portion of the upstream face of the dam. The blanket is loose gravel and hard shale. The plans indicate a 3-foot thick layer of this material has been placed from the toe up to elevation 63.0 (Sheet 3 of Exhibit C1). The riprap has occasionally slipped down the upstream embankment face due to erosion and wave action. This riprap is not adequate for protection of the upstream face.

There is no riprap protection along the stilling basin walls. Bank erosion has been moderate; however, significant erosion could result if the outlet pipe operates at high flows for a long period of time. Rock or bedrock was evident in the bottom of the basin but the actual nature or extent is unknown. Adequate rock material in the bottom will limit downward erosion. At this point, it is assumed that erosion due to high flows would be primarily lateral.

2.4 PROJECT OPERATION AND MAINTENANCE

Jordan Dam is owned and operated by the Arthun brothers. The project was built as an irrigation storage reservoir. There is no formal operation plan, and operation records are not kept.

Storage is dictated by seasonal runoff and supply from a canal which diverts water from the Shields River. The reservoir pool is generally full in the early summer, is lowered during the irrigation season, and refilled during the spring season. Reservoir releases are governed by downstream irrigation requirements, and are provided through a gated 18-inch pipe. Gate control is provided by an operating wheel supported by a concrete pedestal on the upstream face of the dam.

The principal spillway is an unregulated riser. Operation is dependent on the reservoir pool level.

The emergency spillway is of earthen construction and unregulated. Generally speaking, the emergency spillway does not operate on a regular basis. This is due to the small upstream contributory watershed and the fact that the reservoir supply is provided primarily by a diversion canal.

No records of maintenance and repair are available for review. The owner indicates that major problems have not been experienced to date.

Regular organized inspections of Jordan Dam have not historically been performed, and inspection records have not been kept. The project is inspected on an incidental basis when the project operators are at the site to operate the control gate.

There is no formal warning plan of action in the event of dam distress. There are no residences between Jordan Dam and the confluence with the Shields River, which is almost 2 miles downstream. Though not part of a formal plan, the owner indicates that potentially impacted areas along the Shields River would be alerted by radios, phone, or door-to-door contact.

CHAPTER 3
FINDINGS AND RECOMMENDATIONS

3.1 FINDINGS

Visual inspection of the dam, supplemented by analysis of the project in accordance with the guidelines (Ref. 1) and the contract performance standards, resulted in the following findings.

3.1.1 Size, Hazard Classification and Safety Evaluation

In accordance with the inspection guidelines (Ref. 1), Jordan Dam is classified intermediate in size and, based on our visual inspection and judgment, it has a high downstream hazard potential. Therefore, the guidelines' recommended spillway design flood (SDF) for this project is 100 percent of the PMF. Based on reconnaissance level investigations, the project is incapable of controlling the full PMF without overtopping and causing the dam to fail which, in our judgment, would seriously jeopardize life and property downstream. Hence, Jordan Dam does not conform to inspection guidelines (Ref. 1).

3.1.2 Spillway

It appears the spillways have performed satisfactorily in the past. The principal spillway conical trash rack is tilted, but it does not appear to affect the spillway hydraulics. The principal spillway is capable of passing high flows. No detrimental effects would occur except for possible damage in the stilling basin and in the return channel. This particular damage is not considered critical to dam safety unless the high flows are passed for a very long, sustained period of time and damage progresses into the dam embankment. It appears that the principal spillway conduit will operate as a pressure conduit when the reservoir is approximately at or above elevation 62.3 feet. Consequently, it is possible that the conduit would be pressurized for several days while a flood event is being accommodated. Because of this, a pressure conduit could exist within the embankment with no means to facilitate emergency closure (see Recommendation 5).

The emergency spillway is operational and in good condition. The spillway channel is covered with grass, weeds and sagebrush.

Maximum spillway capacity, assuming the reservoir pool is at the overtopping dam crest elevation (67.5 feet, see note on page viii) is approximately 27 cfs for the principal spillway and 11,140 cfs for the emergency spillway (including adjacent

overland flow). The floodwater storage capacity between the principal spillway crest (61.0 feet) and the first overtopping dam crest elevation amounts to 620 AF. In comparison, the PMF for the 3.3 square-mile drainage area is estimated to have a total 72-hour runoff volume of about 2270 AF and a peak flow of 20,720 cfs. Hence, the combination of reservoir storage and spillway discharge capability is inadequate to prevent overtopping of the dam during the SDF.

3.1.3 Low-Level Outlet

It was not possible to inspect the intake structure and the pipe throughout its total length due to the pool level at the time of the survey and lack of appropriate equipment to inspect the small diameter pipe. Wave erosion has uncovered the gate stem pipe and undermined the concrete pedestal. The pedestal is almost completely exposed, and resting on gravel and rock. The gate stem is broken at a threaded connection. This separation has allowed water to enter the cylinder pipe and could effect gate operation in the future. It appears the break resulted from ice pressure and pedestal settlement. At the present time the gate is operational, but it is not possible to determine if a complete gate seal is possible due to the water passing over the principal spillway crest.

3.1.4 Dam

Except for the loss/lack of some riprap on the upstream face and in the stilling basin, and some topsoil sloughing on the downstream face, the Jordan Dam embankment appears to be stable and in good condition. Some questions exist as to the embankment stability because insufficient information is presently available to fully evaluate the dam. Settlement does not appear to be a problem. Seepage was observed throughout the downstream area. The seepage appears to be passing through the abutment or under the dam. The dam drain system does not appear to be functioning properly. The phreatic surface through the dam is unknown. The upstream face of the embankment, the natural shoreline on the right abutment, and the slope in the stilling basin are deficient in riprap protection. Debris on the upstream face of the dam was limited to very small material.

3.1.5 Operation and Maintenance

There are no formal operation plans or maintenance programs, and there are no regular, organized inspections. Project operation is performed according to seasonal factors; that is, seasonal runoff, diversion canal operation, and irrigation water requirements. Maintenance is required on the upstream face of the dam where wave erosion and bank cutting is evident, on the

gate stem casing, and at the downstream end of the outlet pipe where erosion and channel bank sloughing was noted. Jordan Dam is visited on an incidental basis by the project operator to operate the gate controls. There is no formal warning plan of action in the event of dam distress.

3.2 RECOMMENDATIONS

The investigation findings require a high priority be given to the following recommendations:

(1) Immediately develop, implement, and periodically test an emergency warning plan for use in the event of dam distress.

(2) Reshape the upstream face of the dam above the berm and along the natural shoreline of the right abutment to the proper slope. Place a weather-resistant, properly designed, rock riprap material in this area, and in any area below the berm which appears deficient.

(3) Reshape the stilling basin earthen sidewalls and place properly designed rock riprap throughout the basin.

(4) Perform the following work on the low-level outlet: identify the cause of the gate stem casing break and perform the necessary corrective measures; repair the gate stem casing and refill with oil; improve the foundation of the concrete pedestal; inspect the low-level outlet pipe throughout its total length when the pool is at a low level, and make the necessary repairs before raising the pool; check the gate for full operational capability when the principal spillway is not flowing, which would approximate the maximum hydraulic head condition, and when the reservoir pool is at a low level, which will minimize downstream damage; and perform the necessary repairs to the stilling basin (see Recommendation 3).

(5) Inspect the entire 24-inch riser pipe when the pool is at a low level, with special attention to the concrete wearing surface at the bottom of the riser. Provide emergency closure capability under any conditions of the principal spillway conduit at the riser.

(6) Install piezometers in the embankment, abutments, and foundation to evaluate the seepage conditions. Establish a regular monitoring program for the underseepage and abutment seepage. Particular attention should be given to periods when the reservoir pool is relatively high.

(7) Remove the small amount of debris on the upstream face of the dam, particularly near the conical trash rack on the riser.

(8) Repair the small erosion area of the left abutment contact on the downstream side.

(9) Conduct more detailed hydrologic and hydraulic routing studies to better determine the downstream hazard and required spillway capacity and modify the project as studies indicate.

(10) Conduct and place on file a stability analysis of the dam embankment. It is recommended that this analysis be performed by a qualified geotechnical engineer. This study should identify the performance adequacy/inadequacy of the existing embankment drain system. Modify the dam as the study indicates.

(11) Conduct periodic inspections by qualified engineers at least once every five years. Include an inspection of the total length of the low-level outlet pipe and the riser in this program.

(12) Develop and implement a periodic maintenance plan for the dam and appurtenant structures.

Prior to performing engineering studies and remedial construction, coordinate the work with the Montana DNRC, Dam Safety Section, to insure compliance with all pertinent laws and regulations.

REFERENCES

1. U.S. Army Corps of Engineers, Office of the Chief of Engineers Report to the U.S. Congress, National Program of Inspection of Dams, Vol. 1, Appendix D, "Recommended Guidelines for Safety Inspection of Dams". Washington, D.C., Department of the Army, May 1975.

2. U.S. Geological Survey, 7-1/2 Minute Quad Map, Cottonwood Reservoir, Montana, 1972.

3. Montana Water Resources Board, Montana Register of Dams, Inventory Series No. 3, October 1968.

4. U.S. Geological Survey, White Sulphur Springs, Montana, AMS, Scale 1:250,000, 1958 with revision 1972.

5. Jordan Dam, SCS Project Files.

6. Groff, S.L., Reconnaissance Groundwater Studies, Northern Park County, Montana, Montana Bureau of Mines and Geology, Special Publication 2G.

7. Von Hake, C.A., Earthquake Information Bulletin, USGS, Vol. 10, No. 4, 1978.

8. National Oceanic and Atmospheric Administration, Climatological Data, Montana, National Climatic Center, Asheville, N.C.

9. National Oceanic and Atmospheric Administration, Climates of the States, Volume II, "Western States Including Alaska and Hawaii", Port Washington, N.Y., 1974.

10. U.S. Bureau of Reclamation, Design of Small Dams, 2nd. Edition, 1973, Revised Reprint, 1974.

11. Department of Planning and Economic Development, State of Montana, "Comprehensive Area-Wide Water and Sewer Plan, 1970, State of Montana, Park County, 1971.

12. National Weather Service, Interim All-Season Probable Maximum Precipitation Estimates, Missouri River Drainage West of the 105th Meridian, January 1, 1980.

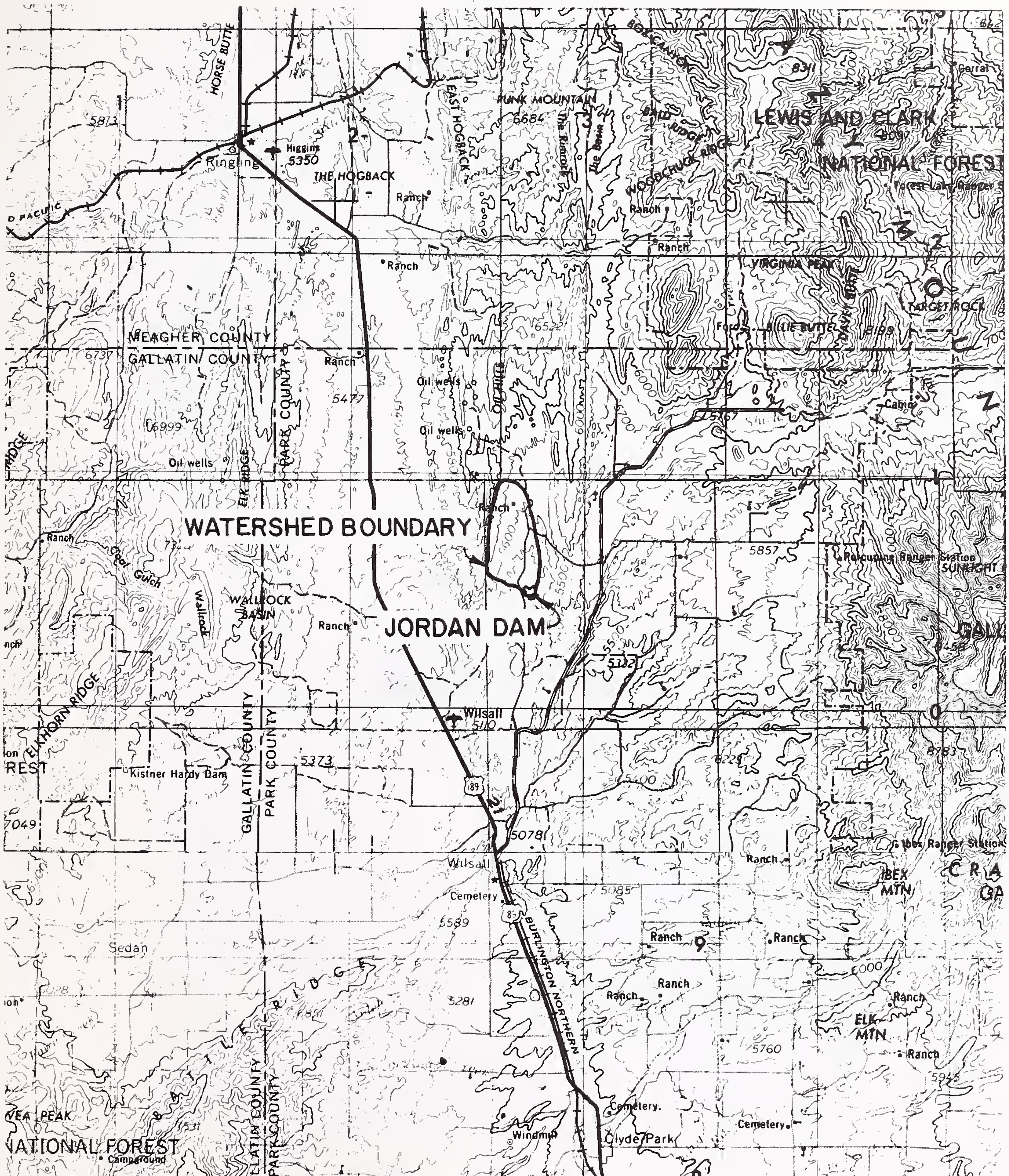
13. National Weather Service, Hydrometeorological Report No. 43, Probable Maximum Precipitation, Northwest States, Washington, D.C., 1966.

14. U.S. Soil Conservation Service, National Engineering Handbook, Section 4, "Hydrology", Washington, D.C., 1969.

15. U.S. Army Corps of Engineers, Hydrologic Engineering Center, HEC-1 Flood Hydrograph Package, Davis, California, September, 1978.

APPENDIX A

VICINITY & WATERSHED MAP



5 0 5 MILES

SOURCE: WHITE SULPHUR SPRINGS AND BOZEMAN, MONTANA, AMS MAP, USGS

N

APPENDIX A

VICINITY & WATERSHED MAP

JORDAN DAM

APPENDIX B

INSPECTION PHOTOS

Photo No. 1 - Drainage Basin.
The drainage basin is composed of flat plains and rolling prairies.

Photo No. 2 - Stilling Basin.
The earthen walls of the stilling basin are vertical due to erosion.

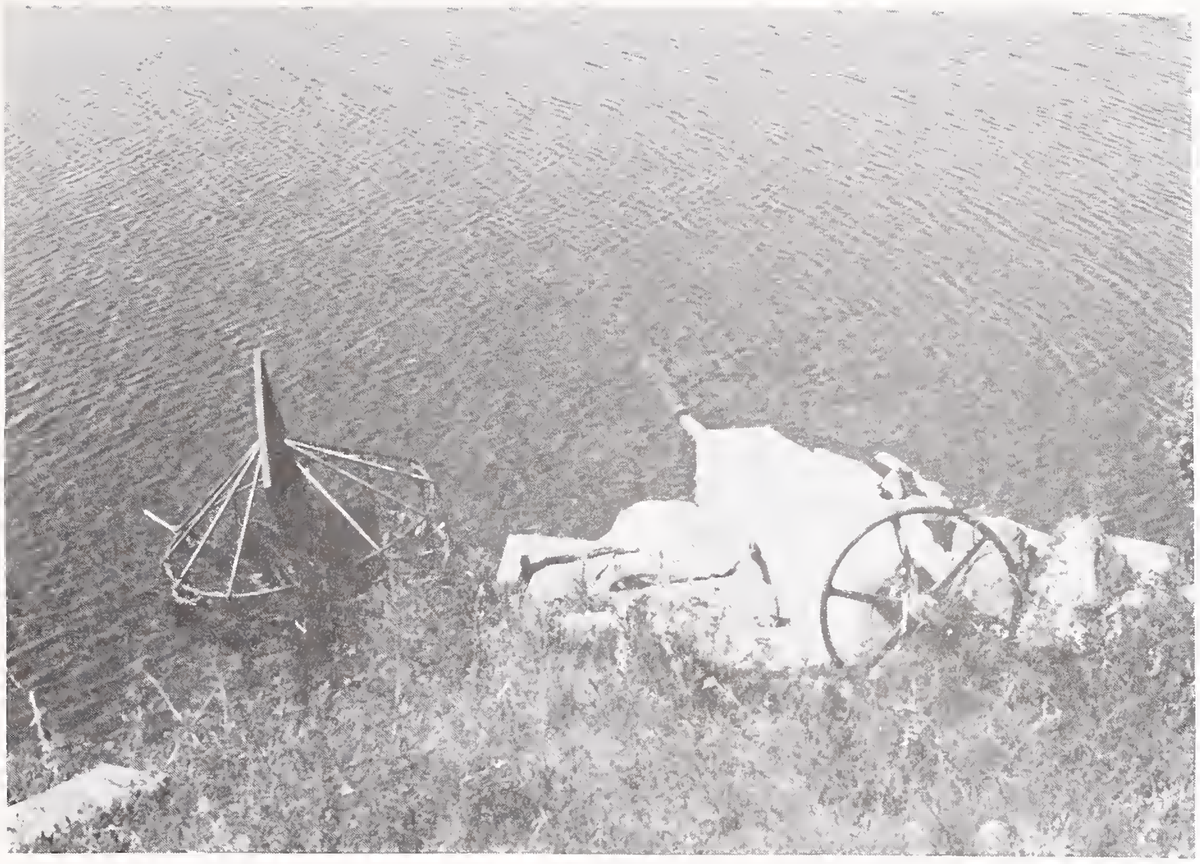


Photo No. 3 - Principal Spillway and Low Level Outlet.
The principal spillway conical trash rack is tilted and the pipe encasing the low level outlet gate stem is broken.

Photo No. 4 - Emergency Spillway.
The emergency spillway is covered with grass, weeds, and sagebrush.

Photo No. 5 - Low Level Outlet.
The pipe encasing the gate stem is broken and allowing water to enter.

Photo No. 6 - Dam Embankment.
The dam crest, shown from the left abutment, is approximately 14 feet wide.

Photo No. 7 - Dam Embankment.
The upstream dam embankment is covered with grass, weeds and sagebrush.

Photo No. 8 - Dam Embankment.
The downstream dam embankment is covered with weeds, sagebrush and grass. Seepage is evident on the left abutment.

Photo No. 9 - Old Dike.
An old earthen dike is located approximately 100 yards downstream of the dam embankment.

Photo No. 10 - Dam Embankment.
Seepage is occurring at the left abutment and is identified on the right side of the photo by the tall grass at the toe of the embankment.

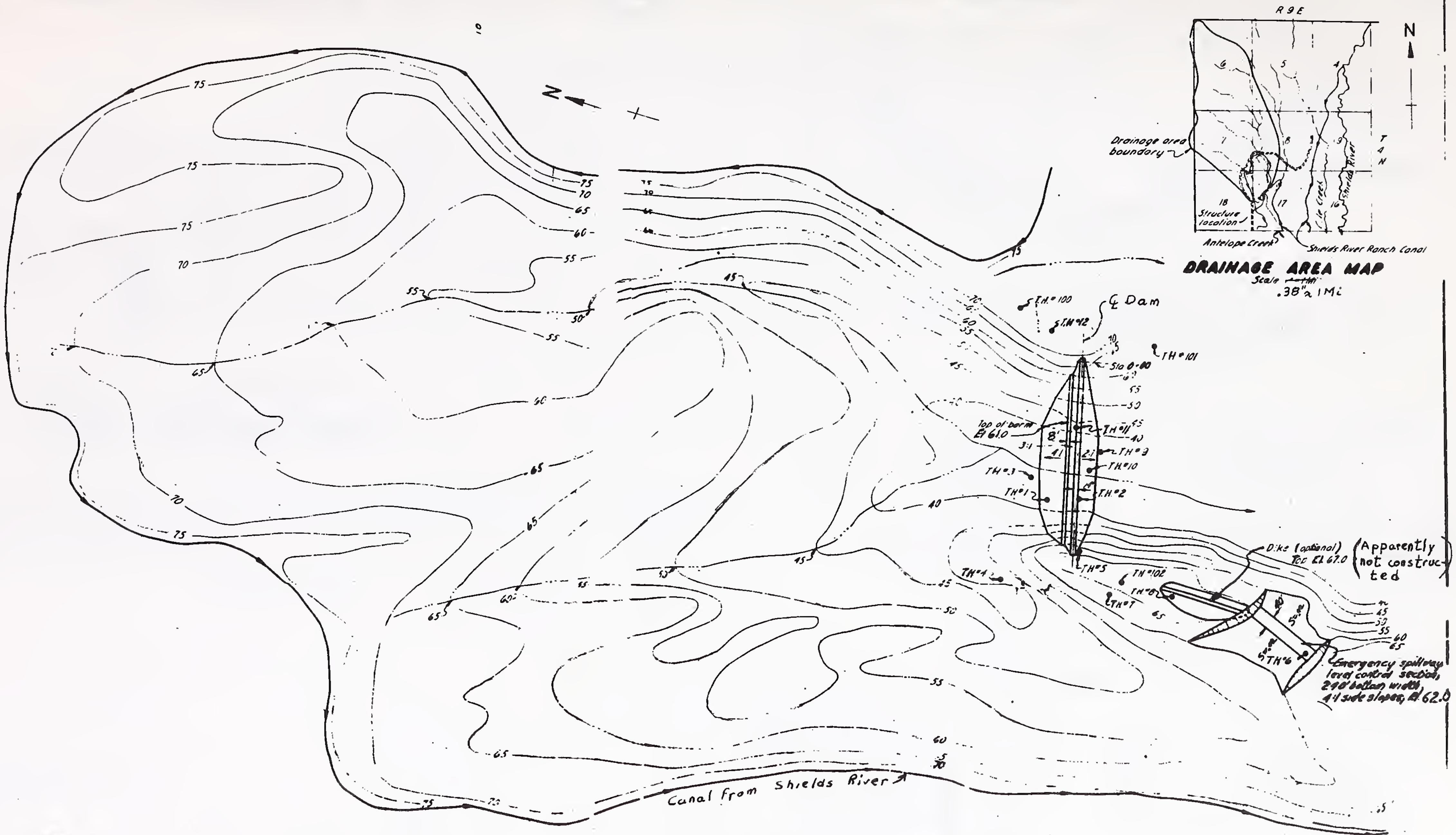
Photo No. 11 - Dam Embankment.
Sections of the upstream dam face have been oversteepened due to wave action.

APPENDIX C

PROJECT DRAWINGS

Exhibit C1	Construction Plans
Exhibit C2	Dam Crest Profile
Exhibit C3	Measured Embankment Slopes

SBM-105
TRASH GAURD



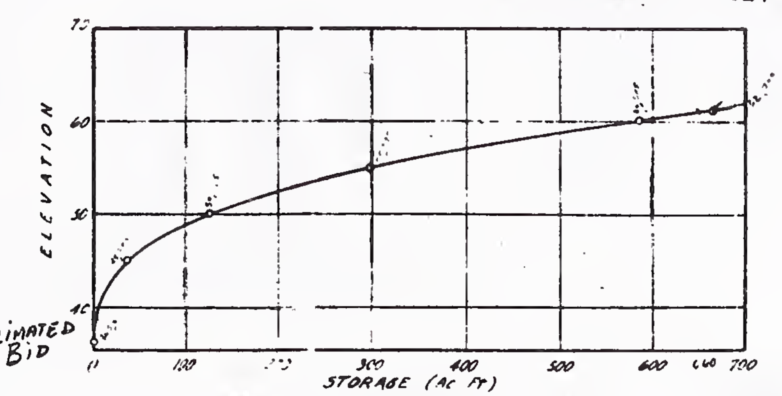
TOPOGRAPHIC MAP OF RESERVOIR
Scale 1" = 524'

TABLE OF QUANTITIES

Adjust To Surface Areas

ITEM NO.	ITEM	UNIT	QUANTITY	ACTUAL
1	EARTH EMBANKMENT (CLASS E-2)	CU.YD.	49,5/5	50,007
2	FOUNDATION STRIPPING	CU.YD.	2,790	3,951
3	CORE TRENCH EXCAVATION	CU.YD.	1,582	—
4	CONCRETE, CLASS 3 (INLET, ANTI-SEEP COLLARS, PIPE SUPPORT, GATE-HOIST)	CU.YD.	6.4	
5	REINFORCING STEEL	LBS.	462	
6	24" DIA. 14 GA. C.M. PIPE RISER, WITH 18" DIA. 14 GA. C.M. PIPE TEES WELDED TO RISER PER DETAIL. ALL PIPE CLOSE RIVETED AND ASPHALT DIPPED	LIN.FT.	26	
7	18" DIA. 14 GA. C.M. PIPE WITH WATERTIGHT CONNECTING BANDS. ALL PIPE TO BE CLOSE RIVETED AND ASPHALT DIPPED	LIN.FT.	194	
8	3 PIECE METAL ELBOW FORM FOR INLET TO 18" DIA. PIPE (PER DETAIL)	EACH	1	
9	CONTROL GATE AND GATE LIFTING MECHANISM CONSISTING OF 18" DIA. METAL SLIDE GATE ARMCO MODEL 35-C30 ~~OR EQUAL~~, APPROXIMATELY 101 FT. OF 1 1/2" DIA. HOT ROLLED GATE STEM, 95 FT. OF 2 1/2" DIA. STANDARD GALV. PIPE, OIL SEAL, AND HANDWHEEL LIFT, ARMCO HB-24 OR EQUAL	LUMP SUM		
10	POSTS, CREOSOTE TREATED WITH PIPE STRAPS, FOR PIPE SUPPORT	EACH	7	
11	FOUNDATION DRAIN WITH 6" DIA. PERFORATED PIPE PER DETAIL	LIN.FT.	265	
12	FOUNDATION DRAIN WITHOUT PIPE (GRADED GRAVEL FILTER PER DETAIL)	LIN.FT.	104	
13	6" DIA. C.M. PIPE, NON-PERFORATED, FOR DRAIN OUTLETS	LIN.FT.	50	
14	TRASH RACKS, ONE EACH FOR RISER AND INLET STRUCTURE [illegible]	LUMP SUM		
~~15~~	~~ROCK RIPRAP, MIN. 12" THICK ON 4:1 SLOPE, DUMPED~~	SQ.YD.	3,855	

ELIMINATED ON BID



STAGE STORAGE CURVE

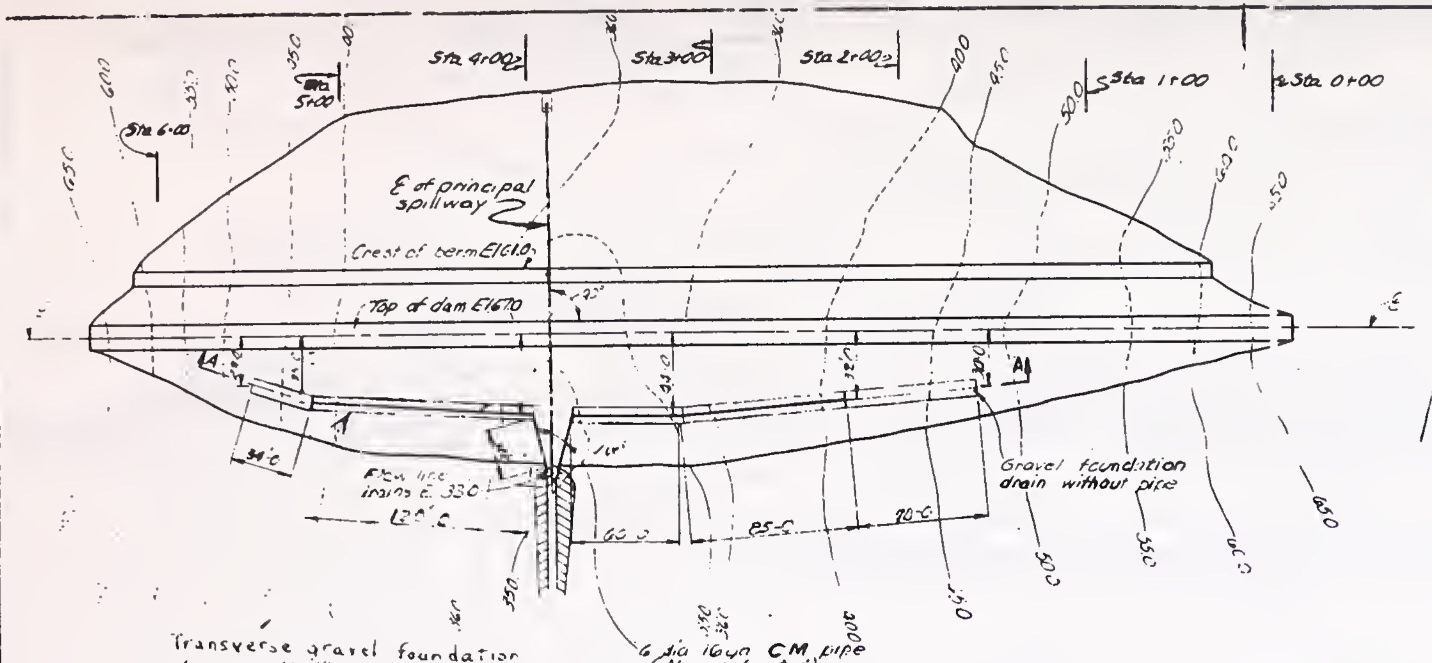
EXHIBIT C1
SHEET 1 OF 4

JORDAN DAM

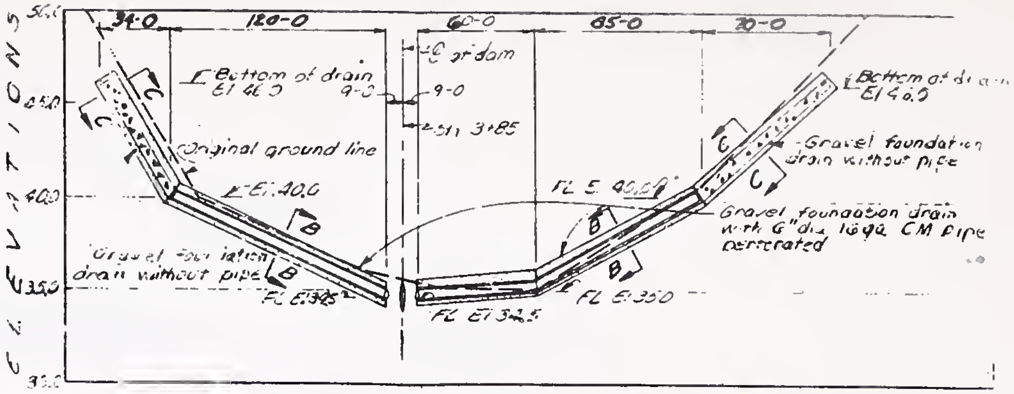
SHIELDS RIVER RANCH IRRIGATION RESERVOIR
NW¼ Sec. 17, T4N, R9E, MPM.
Park County, Montana

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

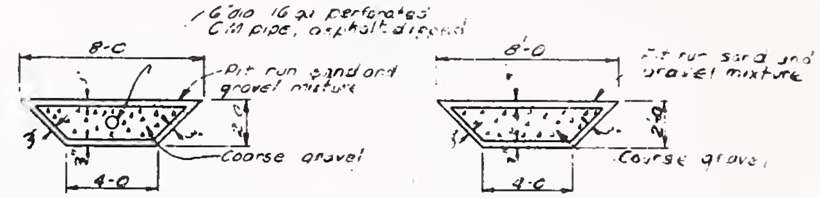
Designed B.B.G., F.W., T.N.	Date 2/61	Approved by E. H. [illegible]	
Drawn R.S.		Title	
Traced		Title	
Checked	S.W.	Sheet No. 1 of 4	Drawing No. 5,E-17,582



PLAN OF BASE OF DAM SHOWING GRAVEL FOUNDATION DRAINS
Scale 1" = 131'



SECTION A-A
PROFILE ALONG CENTERLINE OF TRANSVERSE GRAVEL FOUNDATION DRAINS
Scale: 1" = 131' H, 1" = 13' V



DRAIN WITH PIPE
SECTION B-B

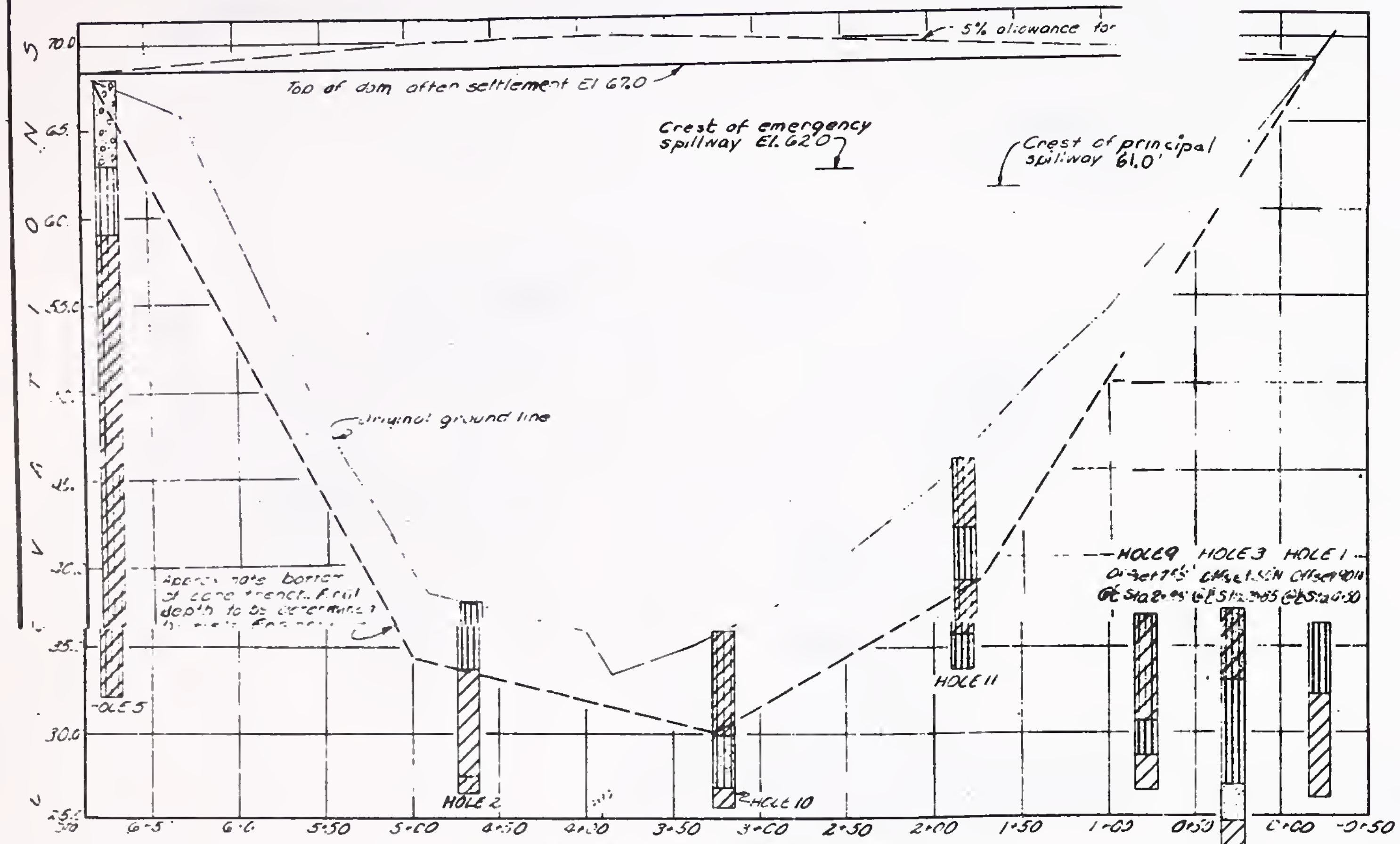
DRAIN WITHOUT PIPE
SECTION C-C

CROSS SECTIONS OF GRAVEL FOUNDATION DRAINS
Scale 1" = 3'-0"

NOTES: 1. Place compacted fill to top of drain where drain is above the original ground and excavate drain in compacted fill.

2. Cover upper ends of CM Pipe with metal covers, lower ends with 1/8" opening screen. Fasten screens and metal covers securely to the pipe.

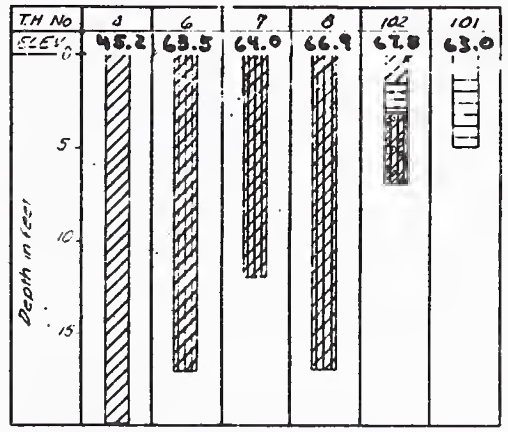
3. Pipe welded at bends in pipe, use connecting bands to join lengths of pipe together.



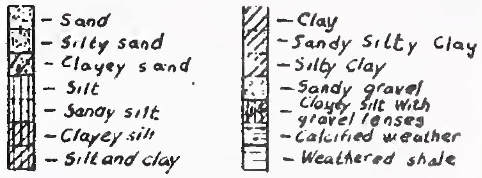
PROFILE ALONG ℄ OF DAM
Scale: 1" = 13.1' V, 1" = 131' H

LOG OF ADDITIONAL TEST HOLES

See topographic map, sheet 1, for location



TEST HOLE LEGEND

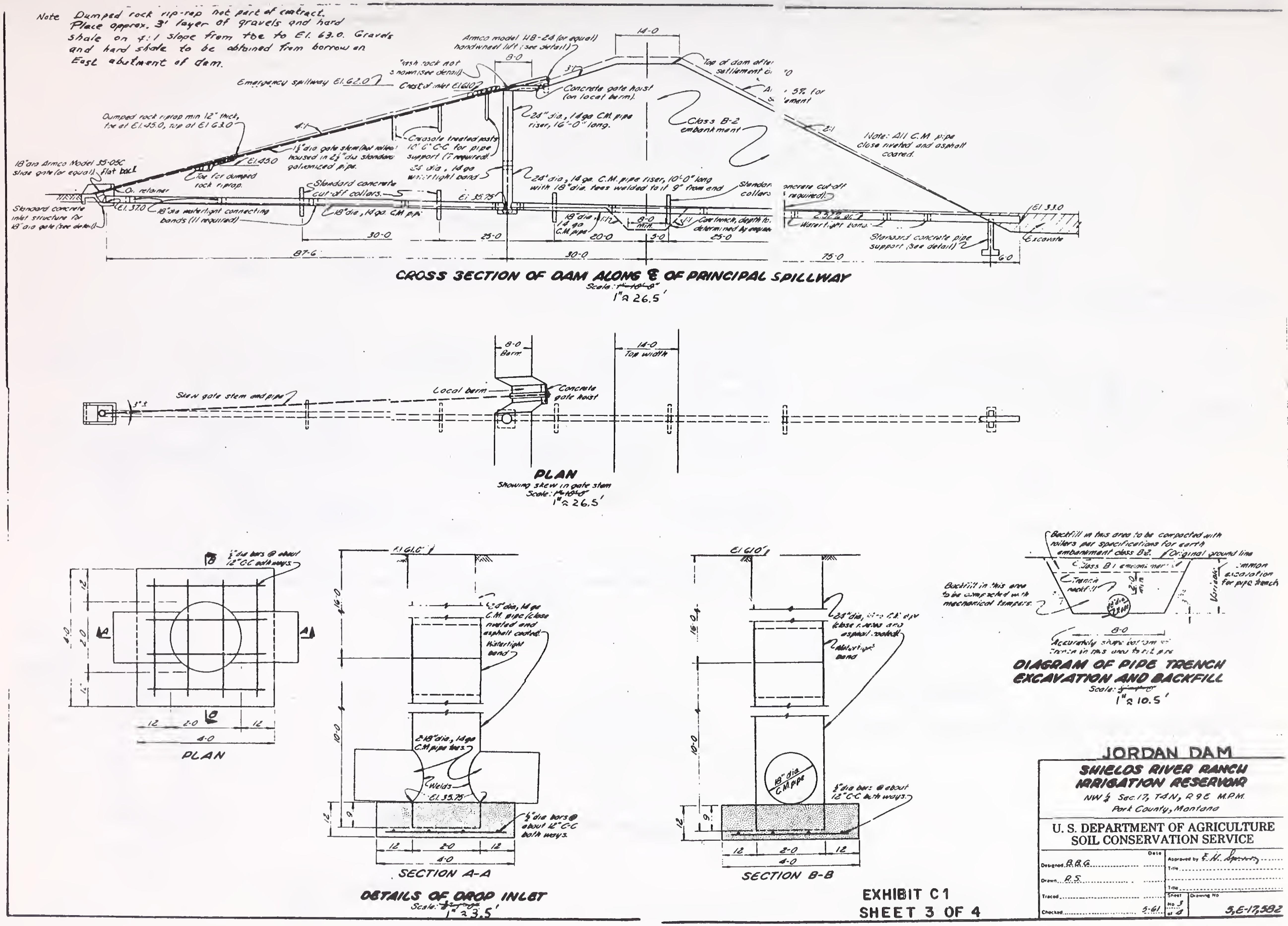


JORDAN DAM
SHIELDS RIVER RANC[H]
IRRIGATION RESERVOI[R]
NW¼ Sec. 12, T 4N R 9E, M.P.M.
Park County, Montana

U. S. DEPARTMENT OF AGRICUL[TURE]
SOIL CONSERVATION SERVIC[E]

Designed B.B.G.	4/61	Approved by
Drawn B.B.G., R.S.	4/61	Title

EXHIBIT C 1
SHEET 2 OF 4



Note Dumped rock rip-rap not part of contract.
Place approx. 3' layer of gravels and hard shale on 4:1 slope from toe to El. 63.0. Gravels and hard shale to be obtained from borrow on East abutment of dam.
Emergency spillway El. 62.0
Crest of inlet El. 61.0
Armco model HB-24 (or equal) handwheel lift (see detail)
Concrete gate hoist (on local berm).
24" dia., 14 ga. C.M. pipe riser, 16'-0" long.
Class B-2 embankment
Top of dam after settlement
Note: All C.M. pipe close riveted and asphalt coated.
Dumped rock riprap min 12" thick, toe at El. 45.0, top at El. 63.0
1½" dia gate stem housed in 2½" dia standard galvanized pipe.
Creosote treated posts 10'-0" C-C for pipe support (7 required)
18" dia Armco Model 35-05C slide gate (or equal) flat back
El. 45.0
Toe for dumped rock riprap.
24" dia., 14 ga. C.M. pipe riser, 10'-0" long with 18" dia. tees welded to it 9" from end
Standard concrete cut-off collars.
El. 35.75
Standard concrete inlet structure for 18" dia gate (see detail)
El. 37.0
18" dia watertight connecting bands (11 required)
18" dia., 14 ga. C.M. pipe
18" dia 14 ga C.M. pipe
Core trench, depth to be determined by engineer
Watertight band
El. 33.0
Standard concrete pipe support (see detail)
Excavate
30-0
25-0
20-0
5-0
25-0
87-6
30-0
75-0
6-0
14-0
8-0
CROSS SECTION OF DAM ALONG ℄ OF PRINCIPAL SPILLWAY
Scale: 1" ≈ 26.5'
8-0 Berm
14-0 Top width
Skew gate stem and pipe
Local berm
Concrete gate hoist
PLAN
Showing skew in gate stem
Scale: 1" ≈ 26.5'
½" dia bars @ about 12" C-C both ways.
PLAN
El. 61.0
24" dia., 14 ga C.M. pipe (close riveted and asphalt coated)
Watertight band
2-18" dia., 14 ga C.M. pipe tees.
Welds
El. 35.75
½" dia bars @ about 12" C-C both ways.
SECTION A-A
DETAILS OF DROP INLET
Scale: 1" ≈ 3.5'
El. 61.0
18" dia. C.M. pipe
SECTION B-B
Backfill in this area to be compacted with rollers per specifications for earth embankment class B2.
Original ground line
Class B1 embankment
Trench backfill
Backfill in this area to be compacted with mechanical tampers.
Common excavation for pipe trench
Accurately shape bottom of trench in this area to fit pipe
DIAGRAM OF PIPE TRENCH EXCAVATION AND BACKFILL
Scale: 1" ≈ 10.5'
EXHIBIT C1
SHEET 3 OF 4
JORDAN DAM
SHIELDS RIVER RANCH IRRIGATION RESERVOIR
NW ¼ Sec.17, T4N, R9E M.P.M.
Park County, Montana
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
Designed B.R.G.
Drawn R.S.
Traced
Checked
Date 5-61
Approved by E.H.
Sheet No. 3 of 4
Drawing No. 5,E-17,582

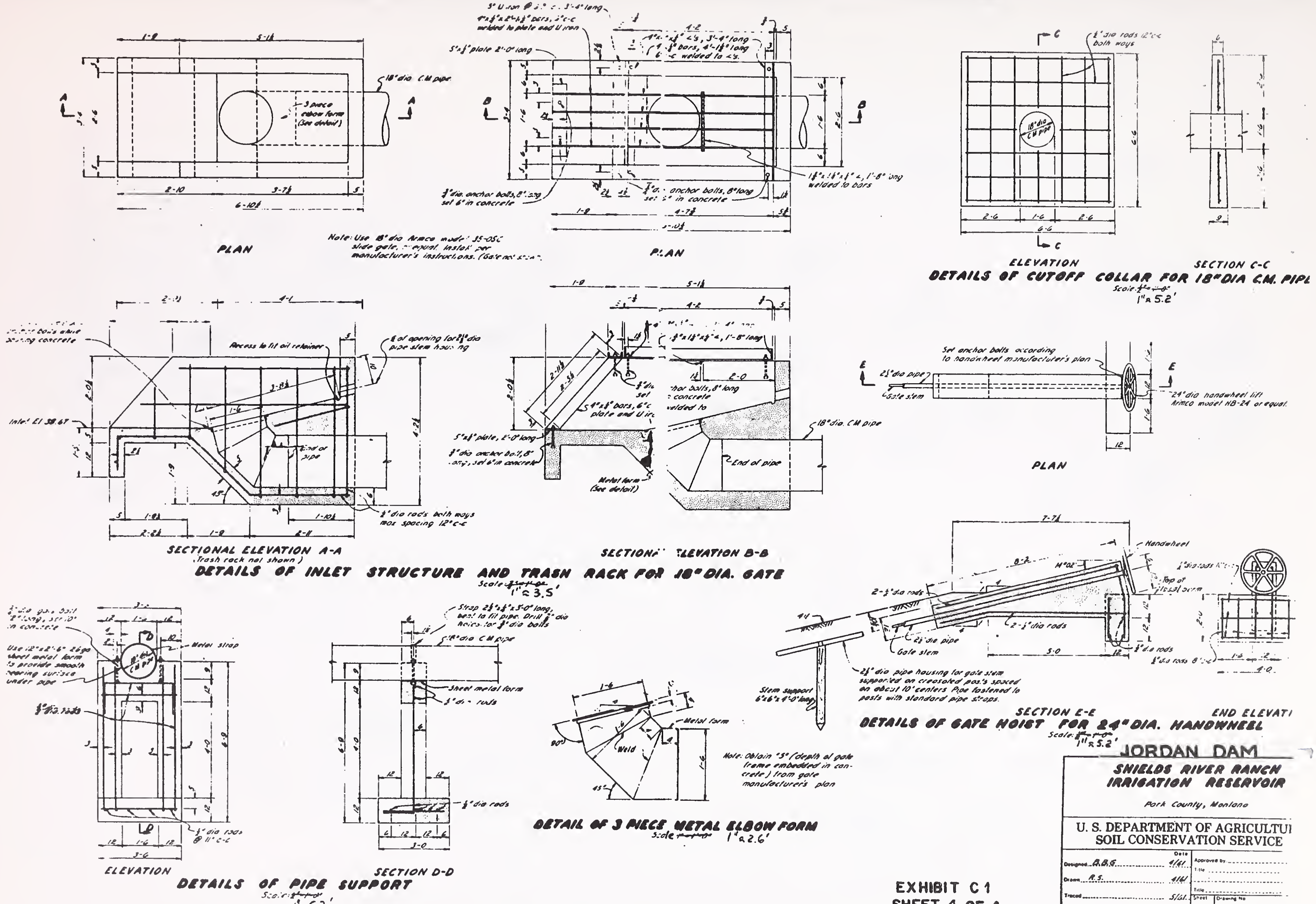
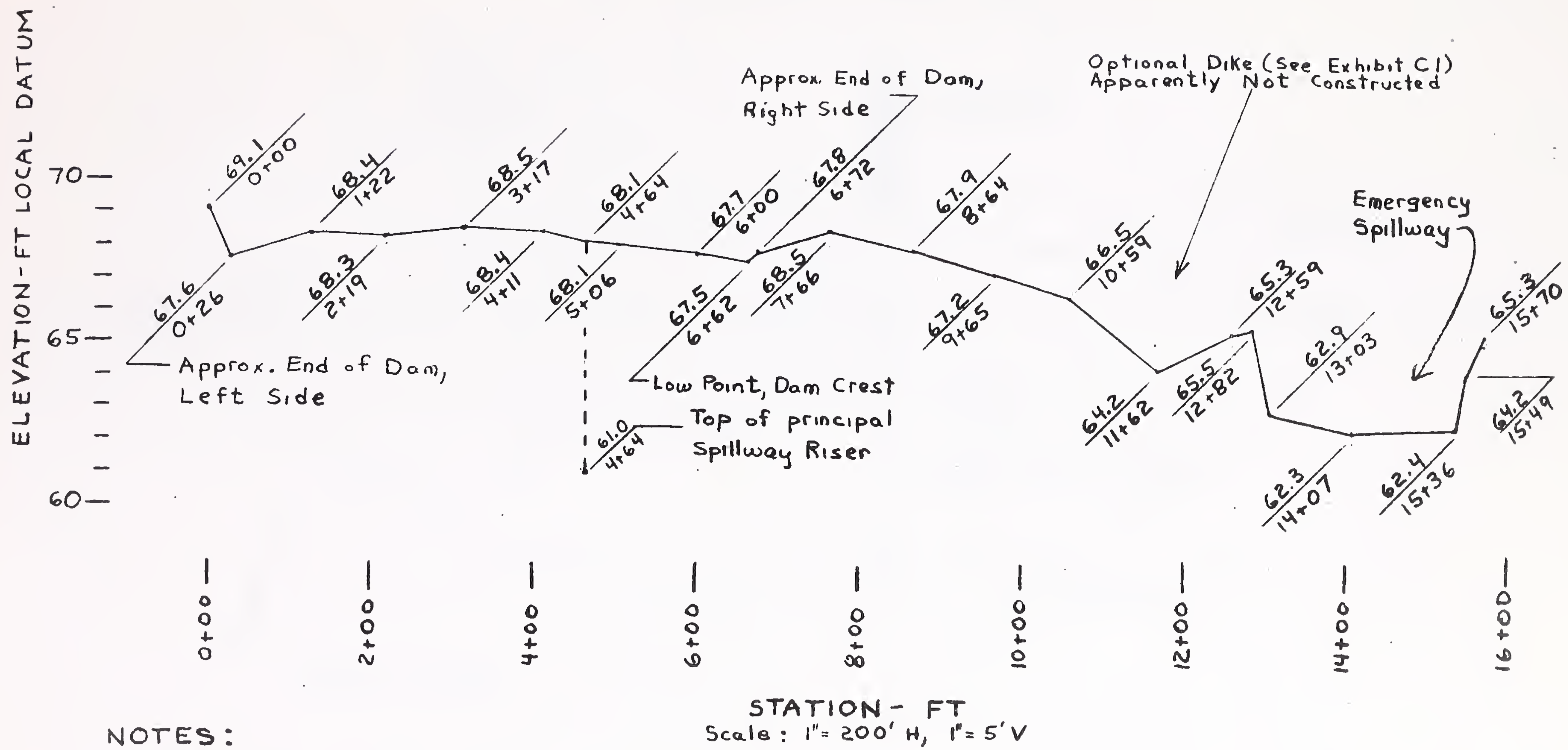


EXHIBIT C1
SHEET 4 OF 4

JORDAN DAM
SHIELDS RIVER RANCH IRRIGATION RESERVOIR
Park County, Montana

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Designed A.B.G.	4/61
Drawn R.S.	4/61
Traced	5/61
Checked	
Drawing No.	5,E-17,5



NOTES:

1. Elevations + stations from field survey taken on 6/26/80
2. Elevations based on top of principal spillway riser = El. 61.0' as shown on the construction drawings

EXHIBIT C2

DAM CREST PROFILE

JORDAN DAM

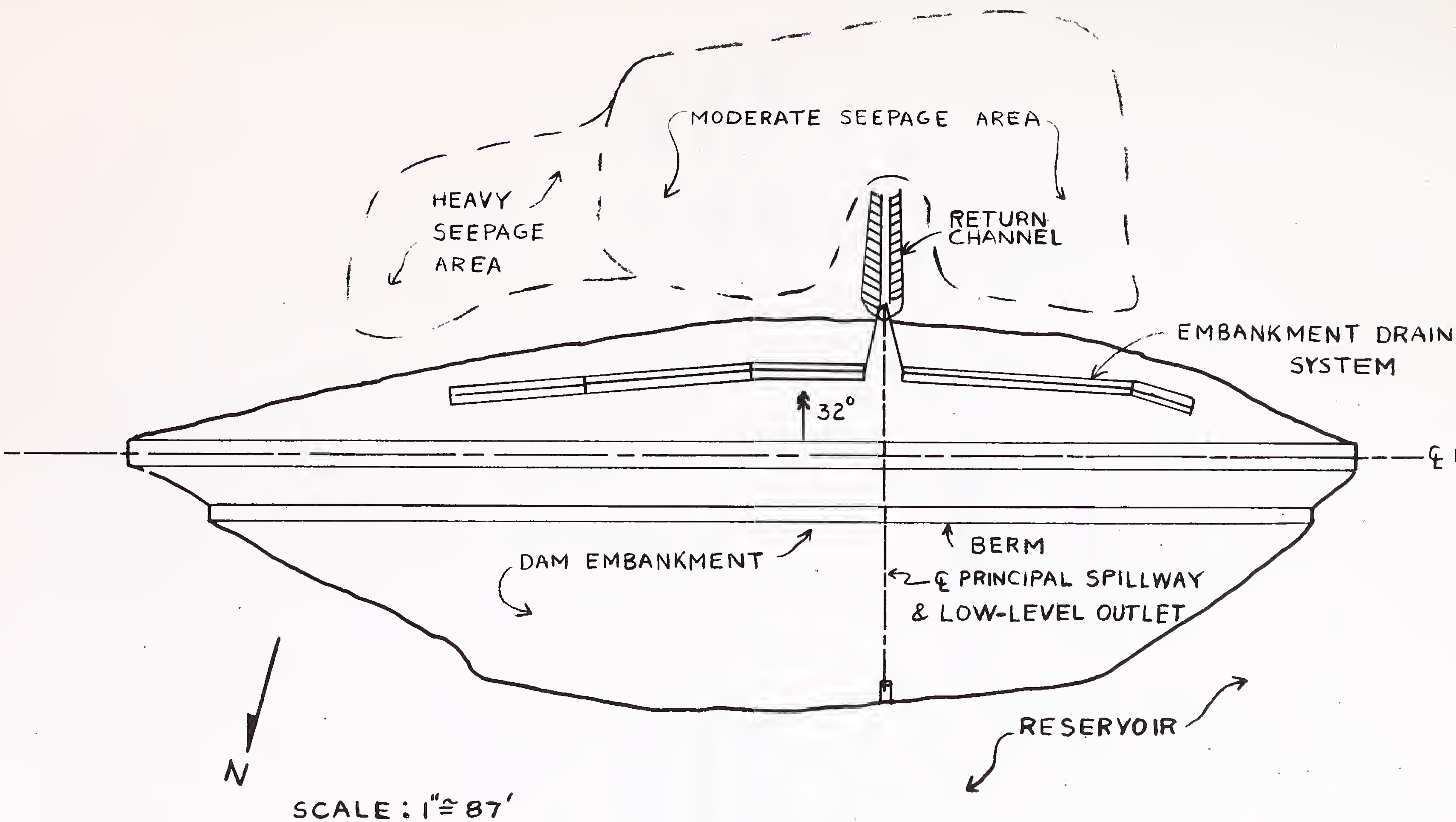
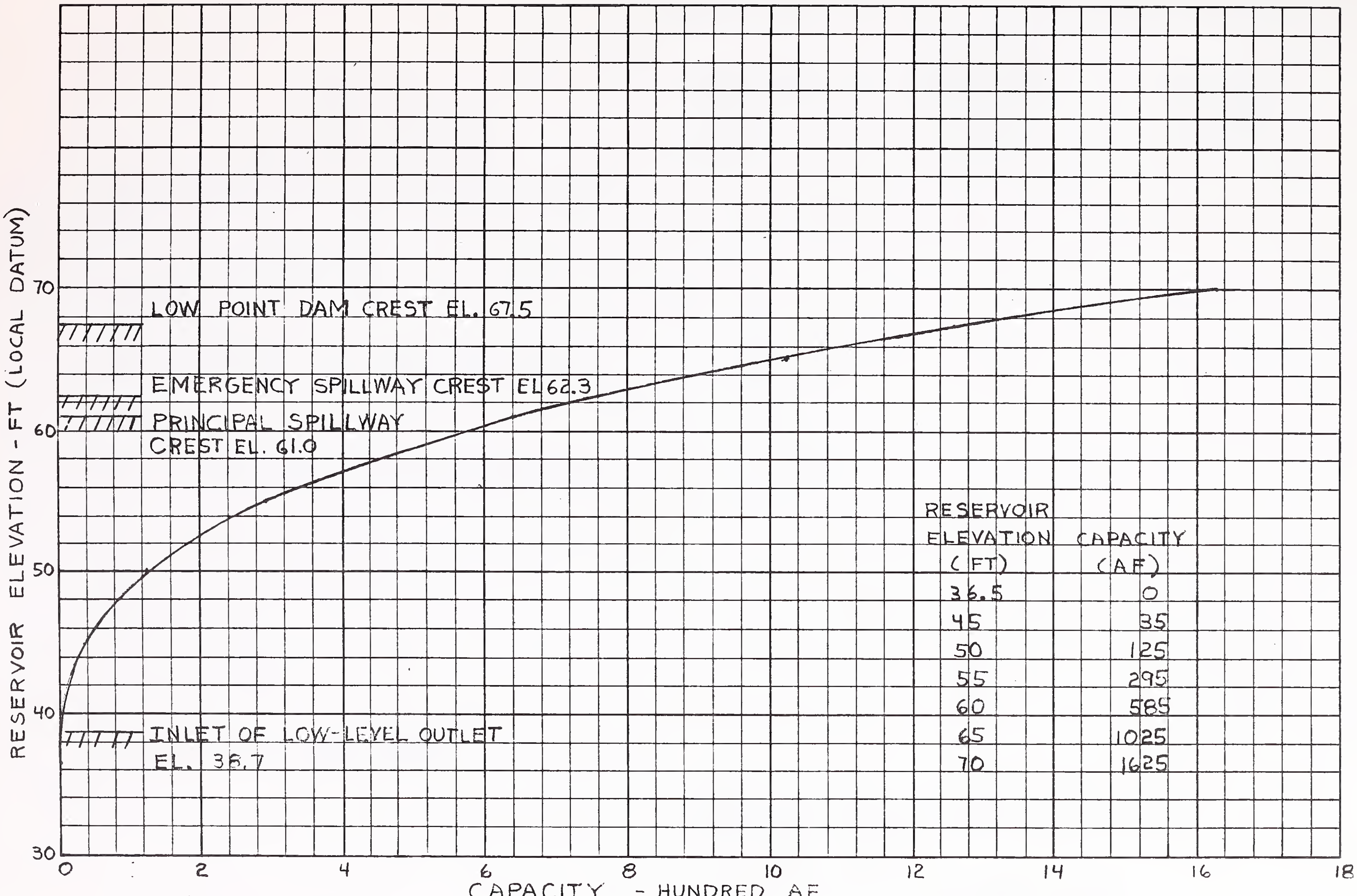


EXHIBIT C3
MEASURED EMBANKMENT SLOPES
JORDAN DAM

APPENDIX D

ENGINEERING DATA

Exhibit D1	Elevation-Capacity Curve
Exhibit D2	Discharge Rating Table
Exhibit D3	Discharge Rating Curves



RESERVOIR ELEVATION (FT)	CAPACITY (AF)
36.5	0
45	35
50	125
55	295
60	585
65	1025
70	1625

SOURCE:
SCS DESIGN FILES

EXHIBIT D1
ELEVATION-CAPACITY CURVE
JORDAN DAM

EXHIBIT D2

DISCHARGE RATING TABLE

JORDAN DAM

RESERVOIR ELEVATION (FT. LOCAL DATUM)	DISCHARGE LOW-LEVEL OUTLET (CFS)	DISCHARGE PRINCIPAL SPILLWAY (CFS)	DISCHARGE EMERGENCY SPILLWAY (CFS)
38.7 (Inlet El. of Low-Level Outlet)	0	-	-
40	8	-	-
50	14	-	-
60	18	-	-
61 (Inlet El. Principal Spillway)	18	0	-
62	*	21	-
62.3 (Inlet El. Emergency Spillway)	*	25	0
63	*	25	200
64	*	25	1400
65	*	26	2900
66	*	26	5540**
67	*	27	9100**
67.5 (Low Pt. Dam Crest)	*	27	11,140**

* Assume above El. 61.0 Ft., Principal Spillway Discharge Governs Through Pipe Common To Low-Level Outlet

** Includes adjacent overland flow

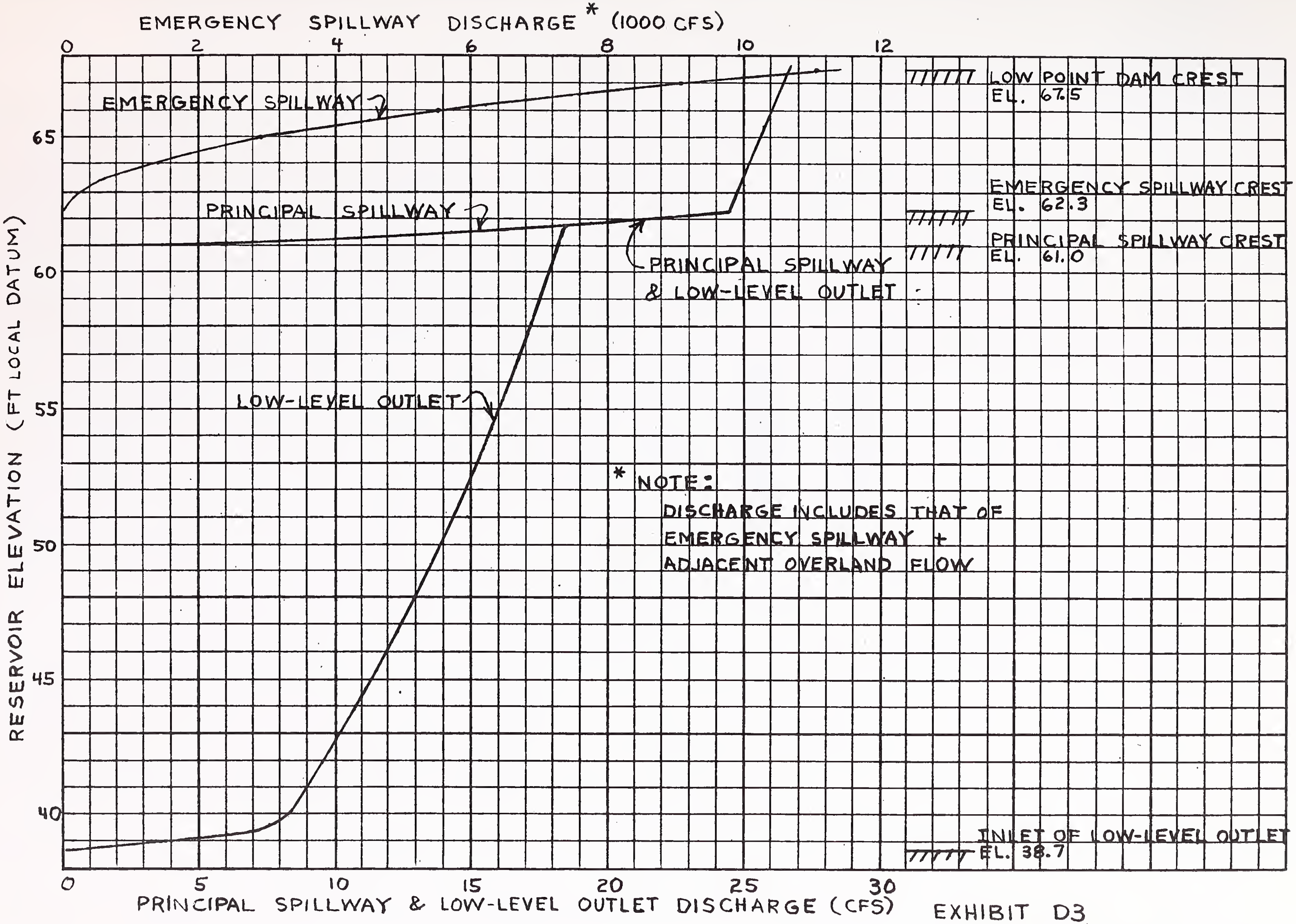


EXHIBIT D3
DISCHARGE RATING CURVES
JORDAN DAM

APPENDIX E

CORRESPONDENCE

DEPARTMENT OF NATURAL RESOURCES
AND CONSERVATION
WATER RESOURCES DIVISION

THOMAS L. JUDGE, GOVERNOR   32 SOUTH EWING

STATE OF MONTANA

(406) 449-2872   HELENA, MONTANA 59601

January 22, 1981

Ralph Morrison
Department of the Army
Seattle District, Corps of Engineers
P.O. Box C-3755
Seattle, Washington 98124

Dear Mr. Morrison:

The Department of Natural Resources and Conservation has reviewed the final draft report on the Jordan Reservoir Dam (MT-334). We concur with the findings and recommendations in the report and feel that the report satisfies the criteria for the Phase I investigation. Our comments have been discussed with your staff and we understand that they will be incorporated into the final report.

Thank you for the opportunity to review and comment on the final report for this project.

Sincerely,

Richard L. Bondy

Richard L. Bondy, P.E.
Chief, Engineering Bureau
(406) 449-2864

RB:LT:lj

United States
Department of
Agriculture

Soil
Conservation
Service

P.O. Box 970
Bozeman, MT
59715

January 20, 1981

Sidney Knutson, P.E.
Assistant Chief
Engineering Division
Seattle District Corps of Engineers
P.O. Box C-3755
Seattle, WA 98124

Dear Mr. Knutson:

Thank you for the opportunity to review the final draft report on Jordan Dam (MT-334).

Our comments relating to specific report statements are:

Page 10, paragraph 4: Twelve in-place density tests were made on the cove backfill and embankment.

Page 17, paragraph 2: As indicated on the plans and design correspondence, gravel and hard shale were substituted for hardrock riprap originally specified on the upstream face. The resulting wave erosion has uncovered the gate stem pipe and undermined the pedestal. Fracture of the stem was likely due to ice pressure and pedestal settlement.

General comments:

We feel the apparent meteorologic and hydrologic conditions used to develop the probable maximum flood are too severe. Were PMP reduction factors considered? Please give the hydrologic conditions used.

Except for the meteorology, hydrology, and possibly the hazard classification, we feel the report is an accurate statement on the dam and its condition.

Sincerely,

Van K Haderlie
State Conservationist

cc:
Ray Smith
Dave Jones

Ronald V. Arthun
Rt. 2
Wilsall, MT 59086
March 13, 1981

Sidney Knutson, P.E.
Dept. of the Army
Seattle District Corps of Engineers
Box C-3755
Seattle, Washington 98124

Dear Mr. Knutson:

I would like to register a protest in regards to your study concerning the safety of the Jordan Dam located at Wilsall, MT. This Dam was built in 1961 by the Shields River Ranch Co. and was inspected on June 26, 1980 by the HKM Associates of Billings, MT.

This dam has been in use for 20 years with no problems and places no hazard on the surrounding area. The dam is fed by a canal, not a live stream, therefore its volume can be controlled as the need arises. This canal would not be affected by the sudden variabilities in weather which can occur.

Personally, I feel that a dam break would cause minimal damage as the volume of water involved would be thoroughly dissipated within a very short distance.

The Jordan Dam was built for agricultural use and is invaluable as a source of irrigation water. Therefore, I feel that your findings are of concern to the landowners who rely on this dam.

Sincerely,

Ronald V. Arthun

Ronald V. Arthun